SERRI Report 81000-01

U.S. Army Engineer Research and Development Center: Rapid Repair of Levee Breaches

SERRI Project: Rapid Repair of Levee Breaches
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U.S. ARMY ENGINEER RESEARCH AND DEVELOPMENT CENTER: RAPID REPAIR OF LEVEE BREACHES

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ACRONYMS

2-D	two-dimensional
ARCH	Arch-Shaped Reusable Cofferdam & Hydrodam
ATD	advanced technology demonstration
CEATI	Canadian Electricity Association Technologies Inc.
DHS	U.S. Department of Homeland Security
DPW	Department of Public Works (ERDC)
ERDC	Engineer Research and Development Center
HPU	hydraulic power unit
HSARPA	DHS Advanced Research Projects Agency
IPET	Interagency Performance Evaluation Task Force
NASA	National Aeronautics and Space Administration
PLUG	portable lightweight ubiquitous gasket
R&D	research and development
RRLB	rapid repair of levee breaches
SAST	Scientific Assessment and Strategy Team
SERRI	Southeast Region Research Initiative

SOUTHEAST REGION RESEARCH INITIATIVE

In 2006, the U.S. Department of Homeland Security commissioned UT-Battelle at the Oak Ridge National Laboratory (ORNL) to establish and manage a program to develop regional systems and solutions to address homeland security issues that can have national implications. The project, called the Southeast Region Research Initiative (SERRI), is intended to combine science and technology with validated operational approaches to address regionally unique requirements and suggest regional solutions with potential national implications. As a principal activity, SERRI will sponsor university research directed toward important homeland security problems of regional and national interest.

SERRI's regional approach capitalizes on the inherent power resident in the southeastern United States. The project partners, ORNL, the Y-12 National Security Complex, the Savannah River National Laboratory, and a host of regional research universities and industrial partners, are all tightly linked to the full spectrum of regional and national research universities and organizations, thus providing a gateway to cutting-edge science and technology unmatched by any other homeland security organization.

As part of its mission, SERRI supports technology transfer and implementation of innovations based upon SERRI-sponsored research to ensure research results are transitioned to useful products and services available to homeland security responders and practitioners.

For more information on SERRI, go to the SERRI Web site: www.serri.org.

EXECUTIVE SUMMARY

In 2007 the U.S. Department of Homeland Security provided initial funding for the development and demonstration of a rapid repair of levee breaches (RRLB) concept. Following successful concept development and testing, funding for the program was continued and has evolved to include multiple components, including the portable lightweight ubiquitous gasket, the rapidly emplaced protection for earthen levees, and the rapidly emplaced hydraulic arch barrier. RRLB devices are primarily tubes made of high strength fabrics designed to be partially filled with water and then floated into a levee breach, which they plug and thus stop or greatly reduce water flow through the breach. Initial studies were accomplished at the facilities of the Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, Mississippi. These were followed by additional large-scale experiments and demonstrations that were successfully completed at the Hydraulic Engineering Research Unit (HERU) in Stillwater, Oklahoma, in September 2008. A series of small-scale experiments conducted in 2008 and 2009 led to the development of concepts of operation for delivering and emplacing RRLB components. In November 2009, the RRLB team completed another round of large-scale experiments and demonstrations at HERU designed to test potential emplacement methods and improvements to previous designs. Following this, efforts were focused on site selection, design, and construction of a large-scale levee breach test facility located at ERDC's Waterways Experiment Station in Vicksburg, Mississippi. The facility is the only one of its kind in the world and will allow researchers to validate results of small- and mid-scale experiments. It could also be used to train RRLB emplacement teams. In December of 2010, a full-scale (40 ft wide) breach with an estimated 2,000 ft^3/s discharge through it was successfully sealed during a public demonstration. The RRLB program is of interest not only to the sponsors but also to state and local government agencies that work in the flood fighting arena. The point of contact for this effort has been Dr. Donald T. Resio, Senior Technologist, CHL, ERDC (Donald.T.Resio@usace.army.mil).

1. INTRODUCTION

The U.S. Department of Homeland Security (DHS) Directorate of Science and Technology is tasked with researching and organizing the scientific, engineering, and technological resources of the United States and leveraging these existing resources into technological tools to help protect the homeland. As part of this task, in 2007 the DHS Advanced Research Projects Agency (HSARPA) and the Southeast Region Research Initiative (SERRI) launched a project to develop and demonstrate concepts for rapid repair of levee breaches (RRLB). In this introduction we examine the motivation for this effort and the clear technological gaps that need to be overcome to succeed in this effort.

Levee breaches can occur very quickly in nature. Because of where they occur and the typical coincident conditions related to the ongoing flooding, they are often very difficult to reach by overland routes. A prime example of the austerity of breach locations in terms of both site access and working conditions can be found in the breach in the 17th Street floodwall during Hurricane Katrina. At this site, the only method deemed feasible for repairing the breach, due to lack of ground accessibility and the high velocities of water moving through the breach, was to drop heavy (2,000 lb) sand bags from a helicopter. This procedure took many days to complete. Because the water level in Lake Pontchartrain remained high for several days after the storm, this allowed huge quantities of additional water to flow into the metropolitan New Orleans area, even after the hurricane had passed. Based on economic analyses conducted by the Interagency Performance Evaluation Task Force (IPET), this rise in water level due to post-storm inflows contributed to about $1.5 billion of additional direct damages.

From this example, we see that simply waiting for flow through breaches to subside or using slow repair methods can be an extremely costly decision in terms of allowing enormous additional direct damages. In this case, the helicopter-lifted sandbags did not seal the breach until after the time that the water levels in Lake Pontchartrain had subsided to a level where significant flow was no longer coming into metropolitan New Orleans.

The need for quickly closing a levee breach is also driven by the potentially fast vertical and lateral growth rates of breaches. As will be shown subsequently in this report, breach widths can grow at rates of tens to hundreds of feet per hour. Such growth rates can turn a small, repairable breach into a large, unrepairable breach in a matter of just a few hours.

From discussions here, we see that the first critical requirement for breach repair must be the time in which a system can be effectively deployed. As a nominal guideline for deployment time, the project team set a 6 h time limit on the total time to deploy an effective rapid levee repair system. This time should include all time following the notice to proceed with the repairs up to the time that flow through the breach is halted. This obviously places some constraints on the availability of a system to deploy in the area where it is needed. This type of a constraint could be met by pre-positioning rapid levee repair systems in an area where they might be needed, either permanently or on a temporary basis.

Given the austerity of the physical settings along most levee areas in terms of land accessibility, which is usually greatly exacerbated during flooding events, it is highly unlikely that levee repair systems that depend on land-based deployments can offer an effective solution in a timely manner. Furthermore, given the relative slowness of travel for ship-borne systems and problems at many sites with accessibility by marine routes (e.g., access to the 17th Street Canal breach and the London Avenue breach by a barge was blocked by debris and other obstructions), deployment by water may be impractical in many situations. These considerations dictate that the optimal deployment should be via airborne lift, with a likely requirement for on-ground personnel for assistance. Implicitly, this places some relatively stringent constraints on the weight of the repair system as practical limits for the lift capacity for helicopters are in the 20,000–30,000 lb range.

A second critical requirement for deployment of a rapid levee repair system is that it be effectively deployable into a breach through which water is still flowing. Flows through breaches

depend primarily on the depth of the breach and the head difference between the water levels on either side of the breach. In extreme situations, velocities in excess of 20 ft/s have been recorded. Based on this, we expect that the transient forces on a rapid levee repair system could potentially be much greater than the static forces. This should not be taken as implying that the static forces will be small, but rather that a levee repair system must be capable of withstanding and supporting very large static and dynamic forces across the breach. Another complication related to actively closing a breach while water is flowing through it is that the shape of the breach periphery can be relatively irregular and can change quickly during the closure process. This introduces some problems with sealing around the edges of a breach if a rigid structure is used for closure.

A third metric affecting deployment is the amount of force that the rapid levee repair system imparts to the levees on either side of the breach. It is highly likely that the levees adjacent to a breach will be compromised somewhat by the same flooding that is causing the breach; therefore, very little reserve holding power may be available in such levees. Because of this, deployment systems and methods which minimize forces on adjacent levees should be considered to be superior to those which do not. From the previous paragraph, we see that this metric will be best met by systems which can distribute the deceleration of the flow over some amount of time rather than instantaneously stopping the flow.

A final metric that will be used here involves a measure of the repair system's complexity. In general, the more complex a system is, the more likely it is to fail. Repair systems that require extensive sequences of operations in series or parallel will always be very difficult to field in severe environmental conditions such as levee breaches. Likewise, a system with many different system components will usually be more prone to failure than a system with only one or two components, simply due to the possibility of failure of any single component and difficulties in mechanically linking system components together in a high-force environment.

We believe that the metrics introduced here represent realistic "stretch" goals for an important problem in the United States today. Having to survive and support very large forces, while still being lightweight, is a definite challenge that had not been met by previous technologies at the outset of this effort. Keeping the system simple and yet with components that are deployable from the air compounded the difficulty of this challenge. However, as will be seen subsequently in this report, having a set of difficult challenges such as these often provides focus for an approach to a solution. It should also be recognized that this project was oriented toward finding a workable concept for RRLB and taking that concept through to a proof of concept stage of development. Since it was not conducted as a slow, deliberate effort to build a definitive foundation for RRLB technologies, it necessarily leaves many ancillary questions unanswered.

Based on a little more than a century of disasters within the United States, it could be argued that flooding caused by hurricanes is the major natural disaster leading to loss of life and property. Figure 1 shows that hurricanes represent a clear and persistent threat to the United States in terms of both loss of life and loss of property. Moreover, inland flooding such as the almost annual floods in the upper Midwest and the potential impact of floods in the Sacramento Delta area and Lake Okeechobee add significantly to this overall threat level. Martin and Olgun (2011), consistent with the information in Figure 1, show that damages related to these natural hazards have continued to rise almost exponentially since the early part of the twentieth century. Given continued sea level rise, even if not enhanced by any changes in climate, and the continuing degradation of natural buffers in coastal areas within the United States, the need for an RRLB solution is expected to continue to grow.

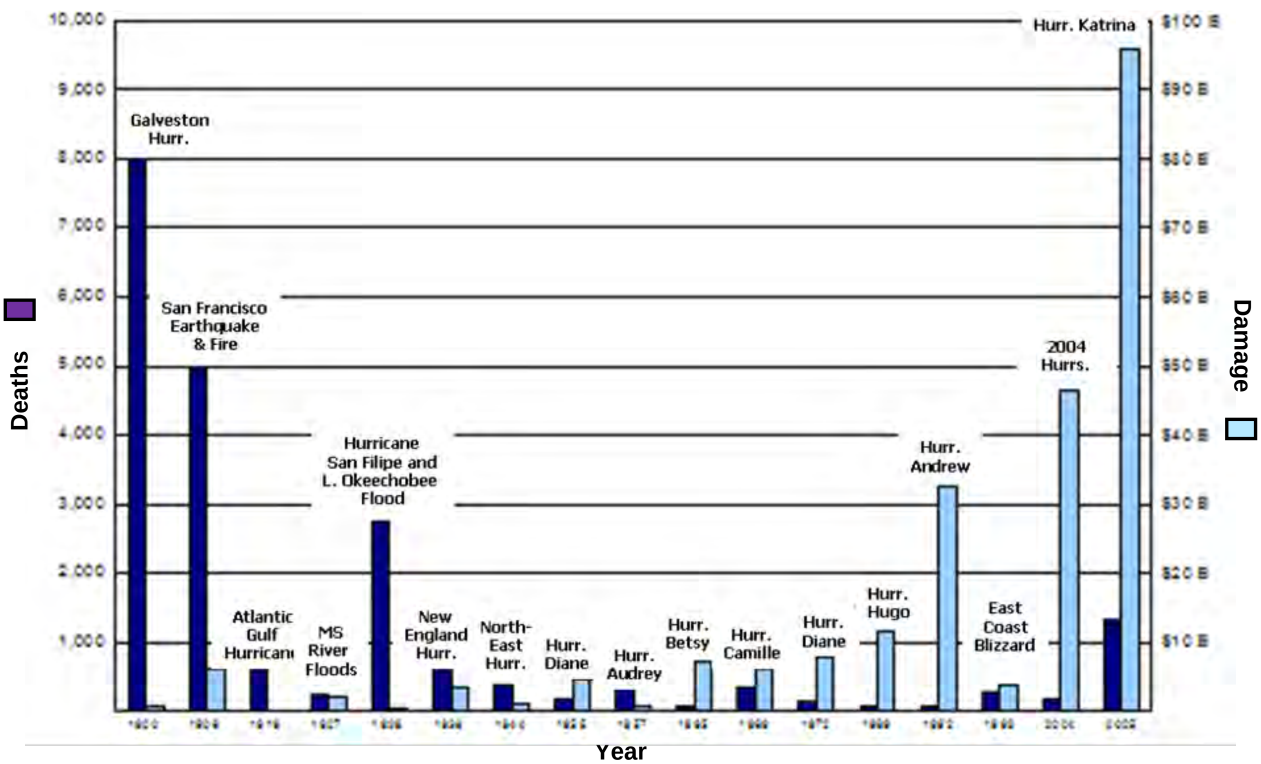


Figure 1.1. Disaster fatalities and property losses in the United States from 1900 through 2005.

The overall RRLB effort was divided into three phases.

1. Phase I, which examined a wide range of innovative concepts for RRLB, down-selected a few and successfully demonstrated these in the Agricultural Research Station facility in Stillwater, Oklahoma, at flows of 125 ft^3/s.
2. Phase II, which developed appropriate deployment methods for the portable lightweight ubiquitous gasket (PLUG) developed in Phase I and made measurements of forces within the PLUG-levee system to confirm theoretical estimates.
3. Phase III, which selected a location for a full-scale test facility, designed a system for full-scale testing at flows of 2,000 ft^3/s, designed and constructed a PLUG to seal a breach 40 ft wide for a flow of 2,000 ft^3/s, and successfully demonstrated this technology.

This report is organized along the same lines as the project was conducted, with each major section describing the primary work conducted in one phase of the effort.

Two separate reports containing supporting technical information are included with this report. The first of these reports was written by Oceaneering International, Incorporated, and provides details on the full-scale model basin, the full-scale prototype test article, and the testing plan. The second of these reports was written by Ward et al. and provides details on specific test conditions, results, and measurements. Each of these reports is intended to be a stand-alone product.

2. INITIAL INVESTIGATION INTO INNOVATIVE CONCEPTS FOR RAPID REPAIR OF LEVEE BREACHES

2.1 Overview of Initial Approach to Rapid Repair of Levee Breaches

This section provides an overview of the different elements of the initial investigation into innovative concepts for RRLB. It provides a foundation for understanding the nature of the problem of levee breach closure and a perspective on the different requirements that must be met to stop the flow through a breach. The theoretical foundations for much of this work are already established, so we will not go into extensive derivations for the equations used in this section; however, we will at least attempt to provide sufficient background to facilitate understanding the basic nature and magnitudes of the forces and problems that must be overcome in the development of effective rapid levee repair technologies.

It is obvious that two critical elements must be met to enable a breach closure. First, the system must be capable of being held in place and not washing through the breach. As an example of this, the weight of the sandbags dropped into the 17th Street Canal breach had to be sufficient to withstand the force of the current passing through the breach without being pushed through the breach. Second, the system must be capable of withstanding the forces acting on it without structural failure, defined here as losing its functionality.

2.1.1 Requirements for Holding a Rapid Repair of Levee Breaches System in Place

The system must be held in place both during emplacement and during the entire interval that it is expected to function in its final position. Means of accomplishing this are expected to involve one or more of the following anchoring methods.

1. Ballast—where the weight of the system or system components is sufficient to resist the local forces acting on it.
2. Anchoring—where the structure is physically connected into the underlying material in the vicinity of the breach.
3. Support from adjacent and underlying levee sections—where remaining levee sections along the sides and bottom of the breach are used to support the system.

As will be seen in the following subsection, loading on anchor locations depends strongly on the number and distribution of “anchoring locations.” We use the term anchoring locations here as we do not want to imply that these are necessarily mechanical anchoring points. It is apparent that each of these methods will have different difficulties and obstacles to overcome and that one method might work best in some areas, while others would work best in different areas. For example, the ballast anchoring method will rely much less on the geotechnical characteristics of material in the vicinity of the breach because it spreads the loading over the entire contact area underlying the ballast element whereas a mechanical anchor, such as one used to anchor ships or a helical anchor, can act more locally on the underlying soils and strata.

It should be noted here that adequate resources to deploy mechanical anchors capable of holding the immense forces associated with the water flowing through the breach might be unavailable. Typical anchors and anchor handling systems on large ships weigh well in excess of the weight constraints imposed by being helicopter transportable. Furthermore, the holding capacity of such anchors varies substantially depending on the material into which these anchors are imbedded and the procedures used to “set” them. Similarly, helical anchors deployed into unknown or poorly known underlying materials may not offer very definitive holding capacities, and in situations where

the currents are very high, such deployments might be extremely daunting if not totally impossible. Thus, the use of mechanical anchors might not be very viable for rapid levee breach closures.

The ballast method offers a somewhat different set of challenges, primarily due to the large amount of weight required to resist the force of water passing through a breach. Although the weights of individual sandbags used in the 17th Street Canal closure were already very unwieldy, it should be recognized that the depth of flow over the sill of this breach during most of the time it was being closed was only about 2 ft. For a large breach with 5–15 ft of flow over the sill, the size of the individual sandbags would become larger than available helicopter lift capacity. The implications of this are that the ballast method may also be quite difficult to implement in many cases, even for simplistic deployment scenarios.

A variation on the ballast concept, developed during the early phases of the present study, was to use water as the primary source of weight for ballast. This theme will be reiterated several times during this report. Because the most available material at the site of a breach is water, it is advantageous to determine ways to effectively use water for as many breach-related purposes as possible rather than attempting to bring different materials to the site. A simple means to provide very substantial ballast at a site is to fill fabric containers with water up to a level where they protrude above the water level in the vicinity of the breach. Water within the water column is essentially neutrally buoyant and contributes nothing to the ballast; however, all water above the surrounding water level contributes directly to the ballast weight. In this case, only the fabric container and the pumps have to be transported, which is quite feasible.

2.1.2 Forces in the Vicinity of a Breach

Let us begin by considering a simple rectangular breach of width W and depth D within a coordinate system with the breach opening aligned along the x axis, the vertical axis being denoted by z, and the axis perpendicular to the breach denoted by y (Figure 2.1). If water were not flowing through the breach, the static force pressure acting at any point within the water in the breach would be given by

$$p(x,z) = \rho g z \quad , \tag{2.1}$$

where

- x is the distance along the breach,
- z is the distance from the surface,
- ρ is the density of water,
- g is the acceleration due to gravity.

The total force for a given depth, D, per unit distance, x, along the breach would be obtained by integration over z and would be given by

$$F_y = \frac{\rho g D^2}{2} \quad , \tag{2.2}$$

where F_y is the total force in the y direction per unit distance along the breach, which shows that the static force is proportional to the square of the depth. Of course the total force acting across the entire width, W, of the breach would be

$$F_{tot} = \frac{\rho g W D^2}{2} \quad , \tag{2.3}$$

where F_{tot} is the total force across the entire breach.

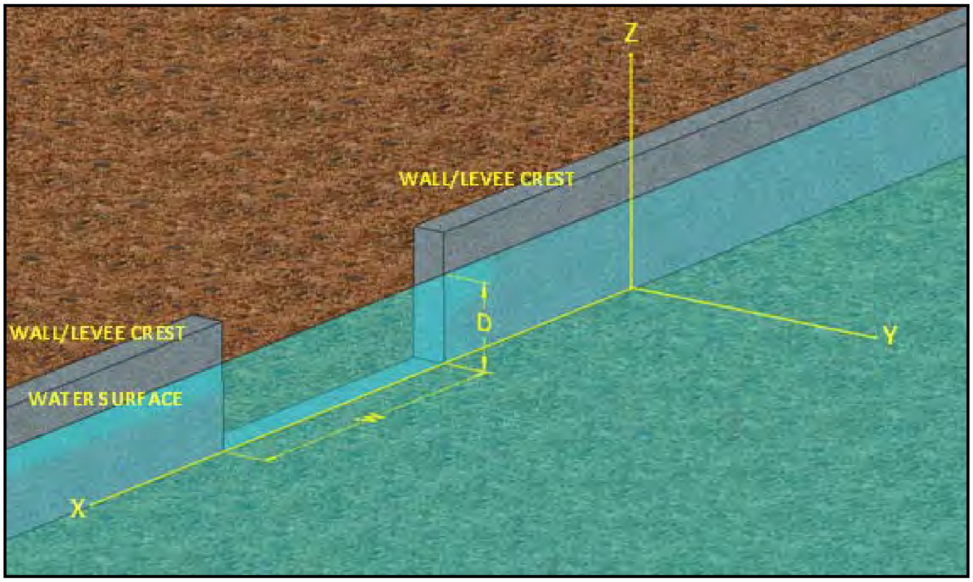


Figure 2.1. Coordinate system used in this report.

This shows that the total force is linearly dependent on the width of the breach. For a beam that spans the width of the breach to carry the static load of the water, it must be capable of carrying the moment generated by this load. The conventional simple form for the bending stress in such a beam is

$$\sigma = \frac{Mr}{I} \quad , \tag{2.4}$$

where

σ is the unit stress per area at the outer fiber of the beam in bending,
M is the bending moment,
r is the distance to the outer fibber from the neutral axis,
I is the moment of inertia of the beam.

Therefore, the beam stress is proportional to the moment carried, which is given by

$$M = \frac{F_{tot}\mathrm{W}}{2} = \frac{\rho g W D^2 W}{4} = \frac{\rho g W^2 D^2}{4} \quad . \tag{2.5}$$

From this equation we see that the moment that must be carried by the beam will be proportional to the breach width squared. Thus, the size of a breach critically influences the ability to span a breach in the absence of intermediate supports. The gist of this is that scale factors are extremely important in consideration of whether or not a concept that works well in a small-scale model is appropriate for prototype-scale applications.

Returning to the issue of the static force on a surface along the breach, we can calculate from Eqs. (2.3)–(2.5) that the total force acting across a breach that is 40 ft wide by 15 ft deep will be about 281,000 pounds. An estimate of the potential dynamic forces on a structure during a closure can be obtained by examining the response of our hypothetical beam to the dynamic shutdown of the flow. As will be discussed later in this report, extreme flows through the breach can reach velocities of 20 ft/s or higher. Recognizing that the total deflection (δz) allowed in a beam spanning a 40 ft breach

will be only on the order of 0.5 ft, the allowable time for deceleration will be about 0.025 s ($\delta z \div V$ or 0.5 ÷ 20), which shows that the sudden insertion of a semirigid structure into the breach would induce very large dynamic loads on the structure. In this case if we take the initial dynamic pressure from Bernoulli's law:

$$p_d = \frac{\rho V^2}{2} \quad , \tag{2.6}$$

where

P_d is the dynamic pressure at the time the structure is emplaced,
ρ is the density of water,
V is the velocity of the current through the breach.

We see that the total dynamic pressure force over the entire breach opening through which water is flowing is, as expected, about the same magnitude as the static force. However, if we attempt to decelerate the flow to zero within 0.025 s, the force on our structure will be about 40 times greater than the static force. This certainly shows that some care must be taken in how quickly we allow a flow to be decelerated in our proposed levee breach closure systems.

2.1.3 Basic Concepts for Stopping the Flow Using Fabrics

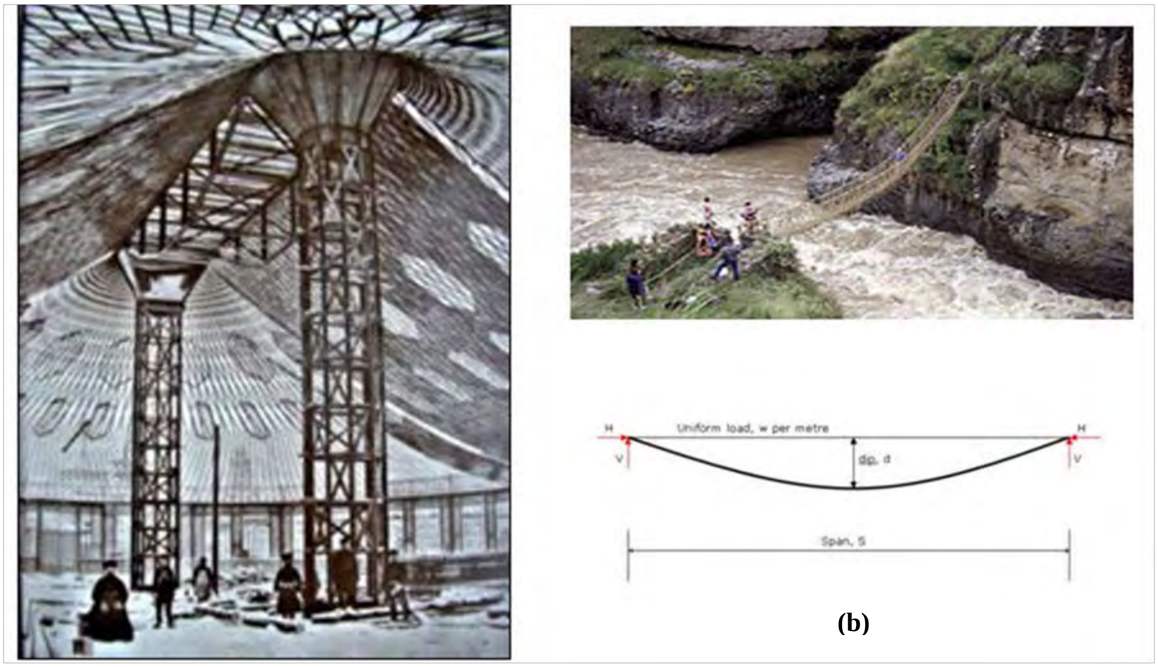
Many alternative methods for rapid levee repair were considered early in our research program; however, we quickly realized that many of these concepts, such as large metal/concrete structures (gated or nongated), would be far too heavy to be airlifted into place. While it might be possible to use barges to transport such structural elements, this is not a very "universal" solution concept, so investigations into this class of structure were abandoned, given the focused nature of our effort. During the various discussions of alternatives during this phase of the project, concepts which included water-filled fabric elements kept emerging as having a very good possibility of success. For the rest of this chapter, therefore, we will address some simple concepts for using fabrics to carry loads in a fashion that might provide a suitable basis for a rapid levee repair system.

Fabrics have been used since ancient times as elements of expedient structures. For the most part, the basic concept has remained the same through the ages—use the fabric to carry tensile loads while allowing rigid (nonfabric) structural elements to bear the compressive loads, as shown in Figure 2.2. In Figure 2.2(a), the structural columns provide the support for the fabrics while in Figure 2.2(b), anchoring the rope into the rock provides the support for the tension.

In the second half of the twentieth century, a new class of fabric structures began to emerge. These structures were based on the pressurization of an enclosed column of air and were typically termed "air beams" or "inflated membrane structures." Many papers have been written on this topic and such structural elements have become an important part of the National Aeronautics and Space Administration's (NASA's) space program, where they afford significant advantages over rigid materials in many of NASA's mission applications. Some contributions to the application of air beams to various types of structures and the theory of their deflections under loads can be found in Bulson (1973); Main, Peterson, and Strauss (1994); Cavallaro, Sadegh, and Quigley (2007); and Wielgosz, Thomas, and Le Van (2008).

Most of NASA's requirements for air beams involved relatively modest forces and did not directly treat forces of the magnitudes required for rapid levee repair systems. Fortunately, the principal investigator for the research and development (R&D) effort described in this report already had extensive experience in the basic extrapolation of this technology to very large forces as the technical manager of an Army-sponsored advanced technology demonstration (ATD). This ATD investigated the ability of a floating beam structure to act as an effective breakwater. Appendix A

provides a brief description of the overall objective and solution methods examined on that project, which ended in a successful final demonstration.



(b)

(a)

Figure 2.2. Examples of fabrics in construction. Oval Pavilion from the 1896 World's Fair (a); using a rope structure to carry a load across a span (b).

For a pressurized tube, the bending under a load can be equated to an equivalent elastic beam [Eq. (2.4)] with the substitution of the beam's equivalent bending resistance term into that equation. A first approximation for this equivalency can be written as

$$EI = \frac{F_b D_t^2}{2\varepsilon_b} \quad , \tag{2.7}$$

where

EI is the equivalent bending resistance,
F_b is the force required to stretch the fabric fibers in the tube to breaking,
D_t is the diameter of the fabric tube,
ε_b is the fractional amount of stretch at the point of breaking.

A similar type of approximation to the allowable moment that can be carried by a pressurized fabric beam before wrinkling (structural failure) is given by

$$M_w = \frac{\pi P_t D_t^3}{8} \quad , \tag{2.8}$$

where

M_w is the maximum moment that can be carried before the tube wrinkles,
P_t is the internal pressure within the tube,
D_t is the diameter of the fabric tube.

Whereas the bending equation for the tube did not have the internal pressure explicitly within it, the equation for the wrinkling moment does. If we assume that the diameter of the tube will scale approximately as the depth of the water, we can rearrange this equation to solve for the pressure required to support the moment generated by the static load across the breach (for the moment neglecting the dynamic load). Unfortunately, this yields an estimate of almost 300 psi. Such an internal pressure would generate a tension around the perimeter of 54,000 lb/in. along the tube. Thus, the tensile breaking strength of a fabric needed to contain such a pressure would have to exceed 54,000 lb for the tube not to explode. Given that this is well beyond present “lightweight” fabric strengths and given that we did not even consider the additional effects of the dynamic forces, which would be much larger on such a rigid tube, this does not seem like a viable alternative for rapid levee repair systems. It should also be noted that the rigidity of the tube would make it very difficult to achieve a good seal along the edges of the breach. Consequently, this idea was abandoned.

Although we knew that it would still be possible to use fabric systems to generate very large ballast for holding rapid levee repair systems in place, we kept looking at new, more innovative methods of using water-filled tubes for this purpose. Finally, we recognized that a major difference between the focus of all of the NASA-funded work on pressurized fabric beams and the work that we were doing was that we were using water, which is essentially incompressible, while NASA research had focused on air, which is quite compressible. This led us to recognize that a new method of using water-filled tubes was possible, one that was based primarily on the resistance of the entire tube to volumetric deformation.

2.2 Breaches in Nature

Both natural and man-made levees have a long history of breaching in nature. Natural levees, built from sediment deposition when rivers overflow their banks, occasionally breach in what is termed a crevasse. Throughout the United States, failures of natural and man-made levees have resulted in lives lost, destroyed infrastructure, and huge economic losses. One example is the part of the Midwest containing the upper Mississippi River, Missouri River, and their tributaries. This area frequently experiences flood events that result in levee failures. Figure 2.3 shows a levee failure on the Pin Oak levee in the Midwest. Figure 2.4 shows the Elm Point levee break, also in the Midwest.

Figure 2.3. Failure of the Pin Oak levee in the Midwest. Note sand bags atop levee used to fight rising water levels.

Figure 2.4. Failure of the Elm Point levee in the Midwest.

Another area where levee failures are of great concern is the Sacramento-San Joaquin River Delta area. Major flood events occurred there in 1950, 1955, 1964, 1986, and 1997. Mount and Twiss (2004) report that the projected subsidence of the delta indicates that it will "become increasingly difficult and expensive to maintain the delta levee system." Some areas of the delta are more than 8 m below sea level. This large amount of subsidence greatly increases the chance of piping-related levee failures, which will be discussed subsequently. Piping-related failures are the major concern in these levees. Figure 2.5 shows the 2004 breach in the levee at the Upper Jones tract in the Sacramento-San Joaquin River Delta.

Figure 2.5. Upper Jones Tract levee breach in the Sacramento-San Joaquin River Delta.

A third area where concerns about breaching are great is the Herbert Hoover Dike around Lake Okeechobee. According to Bromwell, Dean, and Vick (2006), the dike "in its existing condition (1999) is over 4,000 times more likely to fail in any given year from these causes (piping and slope instability) than dams of its kind as a whole." The dike was originally built in response to a hurricane in 1928 that caused loss of life that is second only to the Galveston Hurricane of 1900. Herbert Hoover Dike was originally intended to be a levee and was traditionally viewed as only temporarily

retaining water. It now serves more as a dam. "Herbert Hoover Dike was built from local materials by dredges or draglines without concern for material selection or the nature of the foundation soils (primarily muck and porous limestone) on which it was placed" (Bromwell, Dean, and Vick 2006). Piping-related failures are the major concern at Herbert Hoover Dike.

The current focus on levee breaches was brought about by the large number of levee and floodwall failures that occurred in the New Orleans area in 2005 as a result of Hurricane Katrina. The specifics of the failures are documented in the IPET report (IPET 2006). These failures became the most costly disaster in the history of the United States. During Katrina, levees and floodwalls failed as a result of most of the different causes discussed in this report. Figure 2.6 shows the floodwall failure on the 17th Street Canal.

Figure 2.6. Floodwall failure on 17th Street Canal from Hurricane Katrina in New Orleans.

These examples show that levee breaching and the need to develop methods to rapidly repair levee breaches is a national problem. In the remainder of this chapter we will examine the various characteristics of levees and levee breaches that are important in developing techniques for RRLB.

2.3 Typical Levee Sections

To develop methods for RRLB, an evaluation must be made of the various configurations of levees found in nature. Several typical levee sections at projects throughout the nation are presented in the following paragraphs. Levee height can be defined several ways but height above the landside toe is used here.

Mississippi River. From the headwaters to the Gulf of Mexico, the levees along the Mississippi River vary in size and configuration. Figure 2.7, taken from the Memphis District website, shows a typical levee section in the Memphis District.

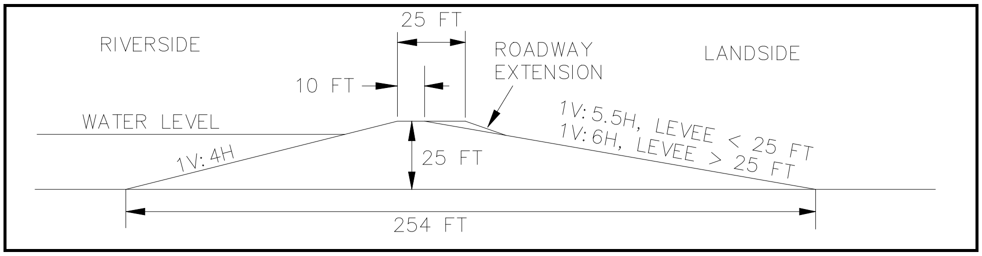


Figure 2.7. Typical levee cross section of the Mississippi River in Memphis District.

Sacramento-San Joaquin River Delta. The levees on the Sacramento-San Joaquin River Delta show wide variation in size and configuration. The *Delta Risk Management Strategy Phase 1; Topical Area: Levee Vulnerability,* Draft 2, Prepared by URS Corporation/Jack R. Benjamin and Associates, Inc. (2007), shows the following ranges of levee characteristics: (a) 7–26 ft levee height relative to landside toe, (b) 1 V : 1 H–1 V : 4.5 H on riverside, (c) 1 V : 1.5 H–1 V : 5.5 H on landside, and (d) 11–38 ft crest width. Using averages of the data, the typical levee section used herein for the Sacramento-San Joaquin River Delta has a height of 18 ft, riverside and landside slopes of 1V : 3H, and crest width of 20 ft as shown in Figure 2.8.

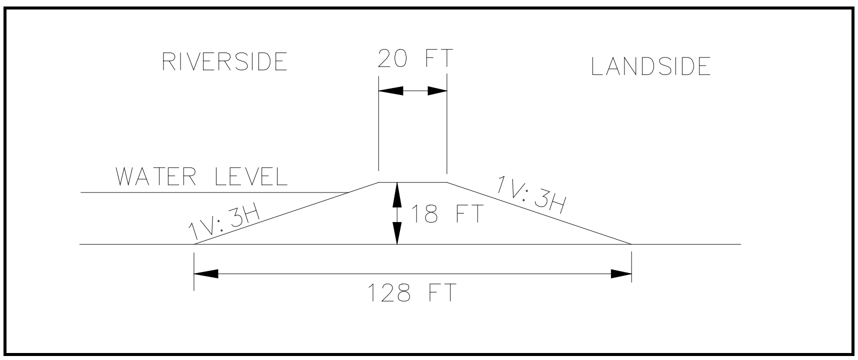


Figure 2.8. Typical levee cross section on Sacramento-San Joaquin River Delta.

Lake Okeechobee/Herbert Hoover Dike. Based on Bromwell, Dean, and Vick (2006), Lake Okeechobbe, Florida, has a crest elevation of 32 to 46 ft with adjacent land elevation of about 10 to 18 ft. Lakeside slopes vary from 1 V : 10 H to 1 V : 3 H, and landside slopes vary from 1 V : 5 H to 1 V : 2 H. Figure 2.9 shows a typical levee section on the Herbert Hoover Dike based on personal communication with Sam Honeycutt of the Jacksonville District.

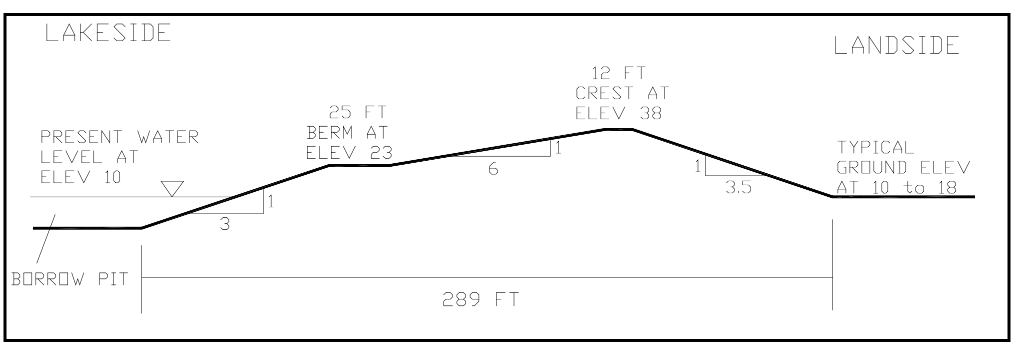


Figure 2.9. Typical levee section on Herbert Hoover Dike.

Lake Pontchartrain at New Orleans. Figure 2.10, based on personal communication with Mr. Ellsworth Pilie of the U.S. Army Corps of Engineers New Orleans District, shows the typical levee section for Jefferson Parish.

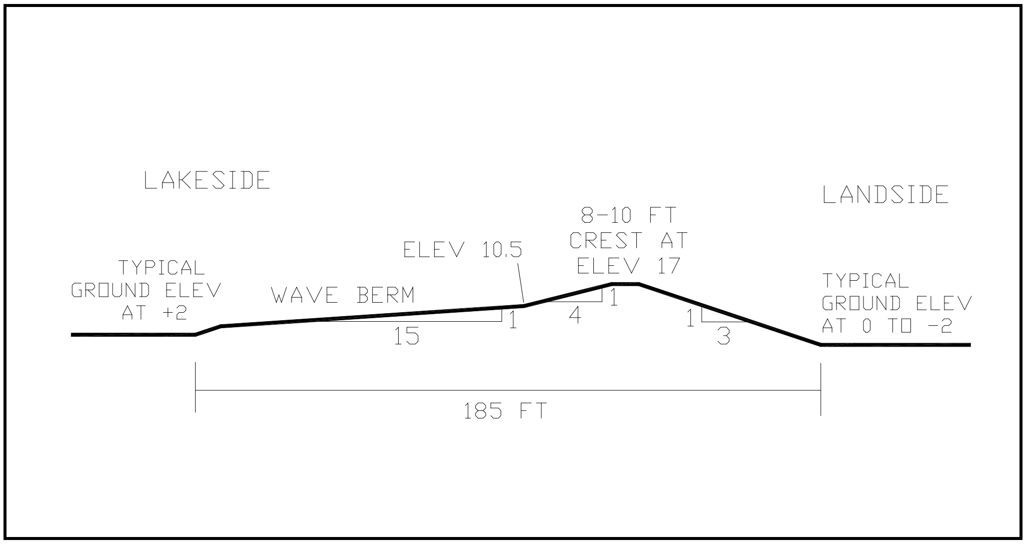


Figure 2.10. Typical levee section on Lake Pontchartrain in Jefferson Parish.

Some of the typical sections presented have changing slope on the upstream face that would be a significant problem for sealing the edges with some rapid repair techniques such as barges sunk over breaches.

2.4 Causes of Levee Breaches

Levee breaches are caused by excessive forces from the water, weakness in the levee material or the levee foundation, or both. Overtopping of levees by floodwater and waves is the most obvious cause. Seepage through or under a levee is less obvious, is far more difficult to predict, and is the major concern in certain areas such as the Sacramento-San Joaquin River Delta and the Herbert Hoover Dike. The various breach causes are discussed in the following paragraphs.

Overtopping- Overtopping occurs when the water level in the river exceeds the crest of the levee or waves spill over the levee. Because of the relatively steep landside slopes of many levees, such as those shown in the previous levee cross sections, once they are overtopped, the water moves rapidly down the land side of the levee. If the height and duration of overtopping is small and the slope is covered by a good layer of grass or other protective material, the levee can survive overtopping. For large heights and durations of overtopping and no landside slope protection, a breach is likely to occur. The overtopping flow will find a locally low or locally weak spot at which erosion is initiated. Once erosion has been initiated, the flow tends to concentrate the erosive forces and the breaching process accelerates. The stages of breach formation will be discussed subsequently. The Scientific Assessment and Strategy Team (SAST) reported that eyewitness accounts indicate that the majority of levee breaches during the 1993 flood on the Missouri River were caused by overtopping (SAST 2007). Wave overtopping and breaching of the levee is similar to overtopping from excessive river water level except that an RRLB technique may have to be placed in waves and withstand wave forces. Figures 2.11 and 2.12 show overtopping over a wide area of a levee along a river.

Figure 2.11. Overtopping of a levee over a wide area with a potential breach growing in foreground.

Figure 2.12. Overtopping of the Foley levee in the Midwest.

Piping/Seepage. According to URS/Benjamin and Associates (2007) in their study of Sacramento-San Joaquin River Delta levees, "80% of the past failures can be attributed to seepage induced failures." Seepage-induced failures are also referred to as internal erosion. Levee seepage is broken into underseepage and through seepage. The SAST report states that these two forms of seepage-induced levee failure occur in equal numbers in the Sacramento-San Joaquin River Delta levees. The SAST report also states "Under-seepage refers to water flowing under the levee in the underlying foundation materials, often emanating from the bottom of the landside slope and ground

surface extending landward from the landside toe of the levee. Through seepage refers to water flowing through the levee prism directly, often emanating from the landside slope of the levee. Both conditions can lead to failures by several mechanisms, including excessive water pressures causing foundation heave and slope instabilities, and immediate and progressive internal erosion, often referred to as piping." The SAST report goes on to state, "Excessive under-seepage is often accompanied by the formation of sand boils. Boils often look like miniature volcanoes, ejecting water and sediments, usually due to high under seepage pressures. These boils can lead to progressive internal erosion, undermining, and levee failure. Boils have been widely observed in all of the historic floods and are believed to have caused significant failures in 1986 and 1997." Through seepage can result in erosion and instability of the landside slope of the levee and lead to a full breach.

Figure 2.13 shows a ring levee of sand bags around a sand boil on the Kaskaskia River levee. Ring levees are one of the existing rapid repair techniques that have been used for many years with great success. Ring levees are placed only to the level that stops movement of sediment with the water. If placed to a greater height to stop flow, the increased head will likely result in the piping connection blowing out somewhere else. Before this picture was taken, the ring levee had reduced the flow and stopped the movement of solid material. Suddenly the levee foundation material started moving again as shown in the figure. Efforts to further raise the sand bag levee and stop the material movement were unsuccessful and the full breach formed as shown in Figure 2.14.

Figure 2.13. Sand bag levee around sand boil on Kaskaskia River.

SAST (2007) reported five factors that contributed to levee breaks in the 1993 flood in the Missouri and Mississippi River: (1) highly permeable substrata; (2) channel banks subject to high energy flow; (3) levee irregularities; (4) inadequate design, construction, and repair; and (5) inadequate levee maintenance. Note that the first item suggests underseepage problems and was based on the observation that 72% of the levee breaks in the 1993 flood were associated with areas occupied by one or more active channels within the past 120 years. This finding is contrary to the previously referenced statement from the SAST report that eyewitnesses reported most failures were due to overtopping.

Figure 2.14. Kaskaskia River levee breach at location of sand boil.

Sliding/foundation stability failure. Although some of these occurred during Hurricane Katrina, foundation stability type failures are infrequent [personal communication, George Sills, U.S. Army Engineer Research and Development Center (ERDC) Geotechnical and Structures Laboratory].

River Currents or Waves Failing Levee Section. Note that in the SAST factors contributing to levee breaks given above, the second item, “channel banks subject to high energy flow,” occurred at the downstream end of bends and channel banks opposite from tributary flows that deflect flow toward the levee. Rapid repair of a levee breach caused by scour of the floodplain adjacent to the levee section could be difficult because of deep depths upstream of the breach, swift currents, and likely loss of a portion of the river side of the levee section for a significant distance on each side of the breach. Nationwide, this is likely a failure mechanism of low frequency of occurrence compared to piping and overtopping.

2.5 Geometric Stages of a Breach

The delineation of geometric stages of a breach may facilitate understanding what RRLB techniques can be used at different stages of breach formation. While the initial stages of the various breach types tend to differ, the latter stages tend to be similar. Stages for overtopping and piping type breaches are defined as follows.

Overtopping. Hanson, Cook, and Hunt (2005) have defined the following four stages of breach formation during overtopping. Note that cohesive embankments fail from overtopping in a series of headcuts on the downstream face whereas noncohesive embankments fail from overtopping by gradual steepening and lowering over most of the downstream face.

1. Stage 1—starts at the beginning of overtopping and ends when erosion of the downstream face has progressed to the downstream edge of the crest. This stage would frequently begin with sheet flow over and down a large length of the levee. The flow would erode a locally weak spot on the downstream face or possibly on the crest. Once

erosion is initiated, turbulence from the eroded area would tend to accelerate the erosion process.

2. Stage 2—starts at the end of stage 1 and ends when erosion has progressed to the upstream edge of the crest. Note that a stage 1 or 2 breach from overtopping has the potential not to result in a full breach if the water level recedes.
3. Stage 3—starts at the end of stage 2 and ends when the embankment has eroded down to the foundation.
4. Stage 4—starts at the end of stage 3 and ends when the breach has finished forming. Stage 4 is the widening phase that is likely accompanied by some and possibly a large amount of deepening to form what are called “blue holes” or “blow holes.” One positive factor regarding scour at the breach is the location of the maximum scour. The deepest scour tends to occur near the landside toe of the levee. Significantly less scour is present adjacent to the upstream toe of the levee. That is advantageous because many of the RRLB techniques are proposed for the upstream side of the levee.

Underseepage and Through Seepage- A comparable set of stages for breach formation from piping is not found in the literature. The stages proposed for piping are as follows.

1. Stage 1—starts when piping is first observed but is not moving material from the levee and ends when material starts being removed from the levee or foundation and begins forming a sand boil.
2. Stage 2—starts at the end of stage 1 and ends upon collapse of the crest. During stage 2 is when a ring levee around the sand boil may be effective in stopping the removal of material from the crest and preventing a breach.
3. Stage 3—starts at the end of stage 2 and ends when the embankment has eroded down to the foundation (similar to stage 3 of overtopping).
4. Stage 4—starts at the end of stage 3 and ends when the breach has finished forming. Stage 4 is the widening phase that is likely accompanied by some and possibly a large amount of deepening to form what are called “blue holes” or “blow holes.” (Same as stage 4 of overtopping.)

2.6 Estimated Breach Formation Time

An analysis was made of data and predictive methods to determine the time required for breach development. Breach development time is needed to determine how much time is available before a breach becomes too large for a rapid repair. While that critical breach size is not known at this time, a breach width on the order of 200 ft (61 m) is presently considered the maximum width that should be considered in the RRLB study.

No full-scale data have been found documenting the formation time of levee breaches. Data are not available to quantify the four stages of geometry of breaches. The largest amount of data and predictive techniques are from breaches of earth embankment dams. Earth embankment dams differ from levees in several ways. One of the most significant is that when a dam is breached, the upstream water level starts to drop and discharge reaches a peak as the breach enlarges and then discharge starts to drop as storage in the reservoir is depleted. Tailwater downstream of an earth embankment dam breach generally has little effect on discharge through the breach or the breach dimensions. Reservoir storage tends to limit the size of the earth dam breach. In most levee breaches, the water level in the river or in a large lake such as Lake Pontchartrain either does not drop or drops only a small amount, and the discharge and breach size continue to increase until the tailwater downstream of the levee

breach rises to reduce and eventually stop the flow through the breach. Tailwater rise tends to limit the size of levee breaches.

Earth embankment dam breaching data are used here and provide the best information on levee breach development times. Because of the limiting effects of tailwater, the data based on dams may overstate the speed of formation of levee breaches. Most of the studies in dam breaching deal with overtopping failures with much less emphasis on piping failures. The Canadian Electricity Association Technologies Inc. (CEATI) Dam Safety Interest Group evaluated models of dam breaching and categorized models for breach formation as empirical, analytical, parametric, and physically based models. Because of the limitations of the first three categories, the focus of dam breach modeling currently being conducted by other researchers is on evaluating several existing physically based models. The analysis presented here to estimate breach development time is based on existing data and empirical methods. This analysis is not an attempt to develop a new empirical approach for dam breaches. Once physically based models have been further developed and validated, more refined estimates of levee breach development time will be available.

Most of the field data on dam breaches presented subsequently do not distinguish between these two times, and the reported time is based on when the breach was first observed until full development of the breach.

Wahl (1998) reports on various empirical relations for breach formation time and presents equations from Von Thun and Gillette (1990) as follows:

$$t_f = \frac{B}{Ch_w} \text{ (erosion resistant, slightly cohesive material)} \tag{2.9}$$

and

$$t_f = \frac{B}{Ch_w + 61.0} \text{ (highly erodible) ,} \tag{2.10}$$

where

t_f is formation time in hours,
B is average breach width in meters,
C is 4 based on Von Thun and Gillette,
h_w is upstream water surface above breach invert in meters.

The Von Thun and Gillette approach was selected for this levee breach time evaluation because it was the only empirical relation relating breach width, breach depth, and breach time and it follows the expected trend of increasing time for increasing breach width and decreasing time for increasing head. Equation (2.10) for highly erodible embankments was not used because the addition of 61 m to the denominator of Eq. (2.10) implies a specific limitation of breach width and height for which it is valid.

Prototype data are presented in Table 2.1 showing breach formation time for various dam breaches. Sources are Zech and Soares-Frazao (2007), which includes the Norwegian tests, the CEATI dam breaching database, and the database in Wahl (1998). Data are limited to dam heights of 30 ft or less to be comparable to most levee heights. The table provides failure cause, breach width, breach depth, failure time, dam composition, the C coefficient based on the Von Thun and Gillette method given by Eq. (2.9), and rate of failure of the breach. The rate of failure magnitude is calculated for both sides of the breach. Some previous presentations of this parameter have calculated a rate of failure for each side of the breach that is 1/2 of the value presented in Table 2.1.

Hanson, Cook, and Hunt (2005) reported on comprehensive large physical model tests of overtopping of cohesive embankments done by the U.S. Department of Agriculture Agricultural Research Service in Stillwater, Oklahoma. The rate of breach widening was observed to be strongly

dependent on the soil material properties. Because these are model test data, the erosion rates are relatively low and their primary value is in validation of physically based models.

Based on Table 2.1, values of C in the Von Thun and Gillette (1990) equation range from 0.7 to 27 with a mean value of 8. The average of the lowest 1/4 of the C values in Table 2.1 is about 2 and should be representative of the more erosion resistant embankments. Because C = 2 is close to the value of 4 adopted by Von Thun and Gillette for erosion resistant embankments, an average value of C = 3 is adopted here to use in Eq. (2.9) for erosion resistant embankments. The average of the highest 1/4 of the C values in Table 2.1 is about 18 and should be representative of the highly erodible embankments. It is obvious that these C values represent a huge simplification of breach formation time and are only presented here to estimate time available to complete a rapid repair.

Table 2.1. Observed data providing breach formation time

Location	Failure cause	Breach width (ft)	Breach depth (ft)	Failure time (h)	Dam composition	C	Failure rate (ft/min)
Impact:							
Norwegian test 1-02	Overtop	69.5 at crest	19.7	0.83–1.17	Homogeneous clay	3.5	0.7
Norwegian test 1B-03	Overtop	65.9 top and base	18.7	0.2	Zoned rockfill	18	5.2
Norwegian test 2C-02	Overtop	33.1	16.4	0.1	Homogeneous gravel	20	6.4
CEATI:							
Glashutte	Overtop	69	26	0.66	Uncertain material properties but compacted	4.0	1.8
Wahl:							
Break Neck Run	?	100	23	3	Rockfill/ earthfill	1.4	0.2
Goose Creek	Overtop	179	13.4	0.5	Earthfill	27	6.0
Grand Rapids	Overtop	62	21	0.5	Earthfill with clay corewall	6	3.0
Ireland No. 5	Piping	44	17	0.5	Homogeneous earthfill	5.2	1.4
Lake Latonka	Piping	129	28.5	3	Homogeneous earthfill	1.5	0.8
Lower Latham	Piping	260	23	1.5	Homogeneous earthfill	7.5	3.8
Oakford Park	Overtop	75	15.1	1	Earthfill with corewall	5	1.2
Pierce Reservoir	Piping	100	28.5	1	Homogeneous earthfill	3.5	1.6
Prospect	Piping	290	14.5	3.5	Homogeneous earthfill	8	3.0
Winston	Overtop	65	20	5	Earthfill with corewall	0.7	0.2

The recommended C values for levee breach times are not recommended for general application to dam breaches. Equation (2.9) is solved for time required to reach a certain breach width for a range of breach heights. Figure 2.15 presents results for erosion resistant embankments, and Figure 2.16 presents results for highly erodible embankments. Figures 2.15 and 2.16 should be used to determine approximate upper and lower bounds for the time available to repair a breach. For

example, a 100 ft wide breach in a 15 ft high levee will form in less than 0.5 h in a highly erodible levee (Figure 2.15) and possibly as long as 3.2 h in an erosion resistant levee (Figure 2.16).

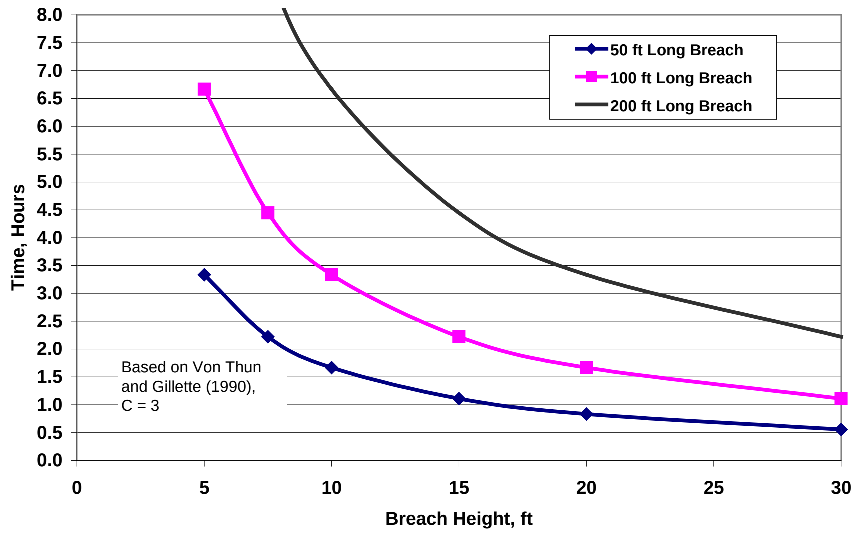


Figure 2.15. Breach time for erosion resistant embankments.

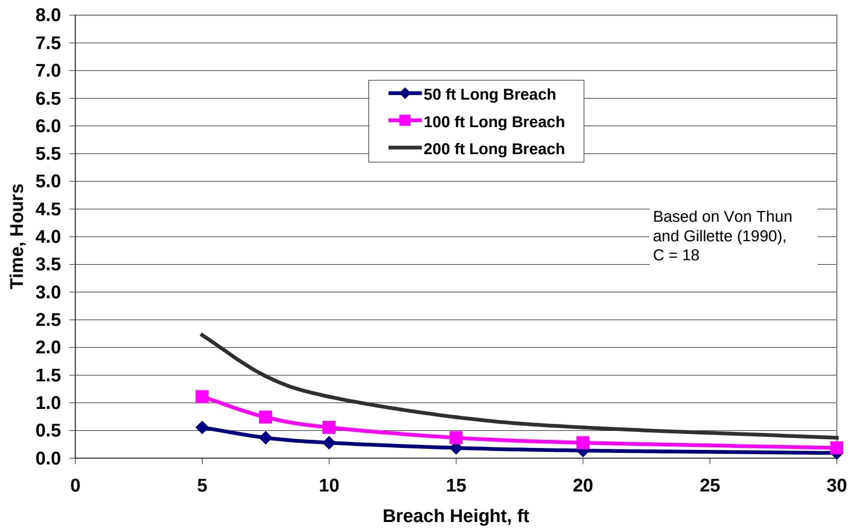


Figure 2.16. Breach time for highly erodible embankments.

Using the simpler approach of omitting the effects of levee height, the average widening rate of the breach ranges from 0.2 to 6.4 ft/min with a mean value of 3.3 ft/min based on the breaches in Table 2.1. The lowest 25% widen at an average rate of 0.5 ft/min, and the highest 25% widen at an average rate of 5 ft/min.

2.7 DISCHARGES THROUGH A BREACH

2.7.1 Numerical Simulation

This section documents a numerical simulation of discharge through a levee breach. The intent was to simulate a worst case scenario of discharge through a breach to determine the maximum forces that might be exerted on any object placed in the breach. To determine the dynamic hydraulic forces produced by the discharge through the breach, the velocity of the flow must be known. It is expected that the highest velocity will produce the greatest dynamic hydraulic force. Thus a major goal of the numerical simulation was to map the velocity through the breach in both time and space.

Normally a breach forms slowly by overtopping, scouring, or piping; but that is not always the case as exemplified by the failure of the Taum Sauk Upper Dam, Missouri, in December of 2005. Figure 2.17 shows a general view of the Taum Sauk Upper Reservoir near Ironton, Missouri, just after the breach.

Figure 2.17. Aerial photo of the Taum Sauk Upper Reservoir just after breach in December 2005.

Figure 2.18 shows a closer view of the emptied lake and breach vicinity. In this case the lake wall was overtopped and also suspected of being undermined by seepage. When it collapsed, the lake was emptied in a matter of 12 to 15 min. It is suspected that the 17th Street breach in New Orleans failed catastrophically after some initial period of overtopping as well. Knowing that such documented cases exist, for this study a worst case scenario was considered as an instantaneous removal of the levee wall, allowing flow to cascade out of the breach.

Figure 2.18. View of emptied lake and vicinity.

The conceptual model was staged as water flowing down the Mississippi River. A levee with dimensions similar to large Mississippi River levees is on one side of the model. Opposite the levee is a basin into which the flood waters will flow. Figure 2.19 shows this situation.

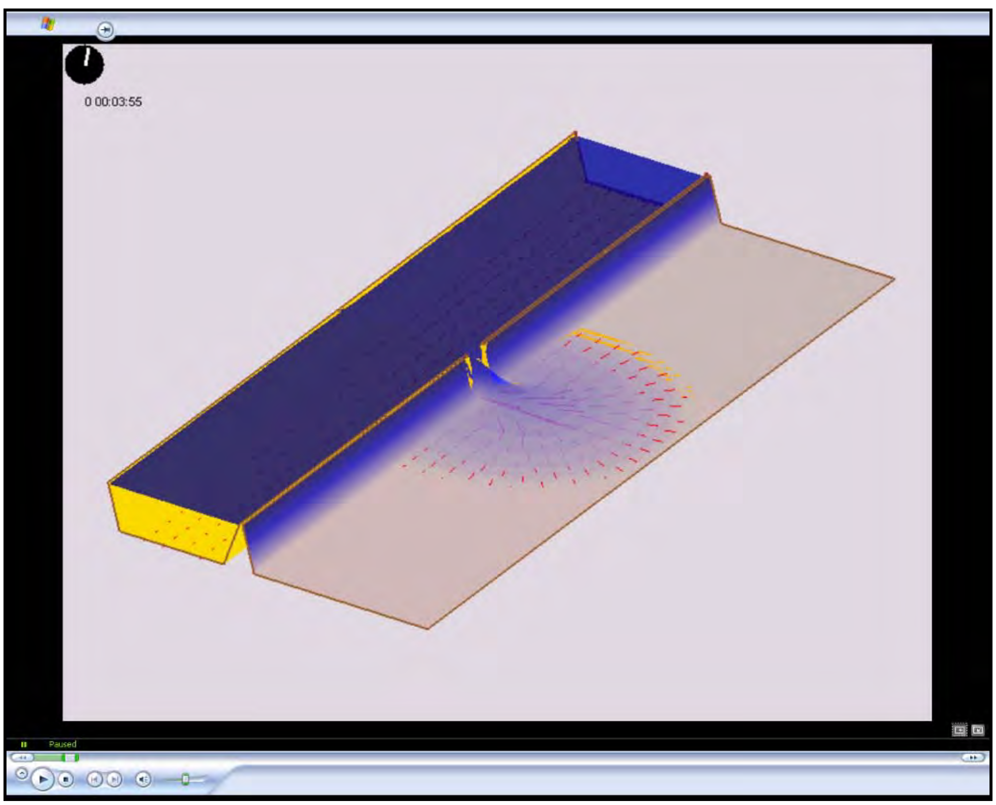


Figure 2.19. Schematic diagram of computer model of idealized breach.

The model is 6,200 ft in length and about 2,760 ft wide. In the figure the left side represents the river and the right side with the grey-brown floor represents an empty and dry basin into which the water from the breach will flow. The bottom of the simulated river and the ground elevation of the basin are made the same in this model. The initial simulation conditions were set with appropriate discharge in the river to maintain water at a depth of about 24.5 ft. The simulation starts when a 200 ft section of the levee is instantaneously removed. Water falls 24.5 ft across the breach onto the dry ground of the basin. The numerical grid for this simulation is shown in Figure 2.20 as an oblique view with a 5 V to 1 H distortion. There are 14,134 two-dimensional (2-D) triangular elements in the grid and 7,369 nodes.

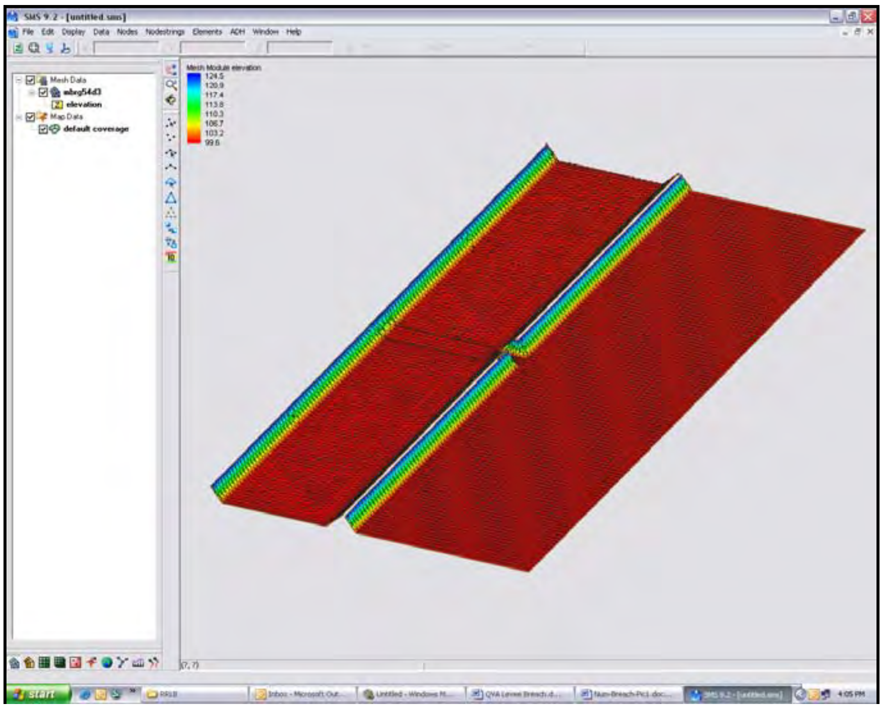


Figure 2.20. Illustration of nodes used in the idealized breach simulation.

Flow is into the river at the top and out at the bottom, with lateral flow into the basin. The basin is fully enclosed so as to model the effect of filling on the breach discharge. Figure 2.21 shows a closer view of the 200 ft wide breach opening in the levee wall.

The simulation is noteworthy because the simulation conditions are very extreme. Water will flow from subcritical to supercritical over a large area in an extremely short time, over a relatively large area, and onto a dry surface. Thus any model used to simulate the scenario must be able to model flow transitions, possibly shock capturing, and cell wetting and drying. The ADaptive Hydraulics 2-D model (also referred to as ADH 2D) was selected because of its purported capabilities in these areas.

The simulation time from the initial time of the breach until the basin was filled was about 2 h. Required computation time was about 12.5 days on a PC with a Pentium(R) 4, 3.2 GHz, CPU and 1 GB of RAM. At several stages of the simulation, time steps were cut to fractions of a second to facilitate convergence. Also the adaptive capability of the model was invoked and necessary. At the initial time of the simulation run, it was not known how long it might take or whether it would converge. The run time was very long, and future runs of this type should be done on faster, multiple processor computers.

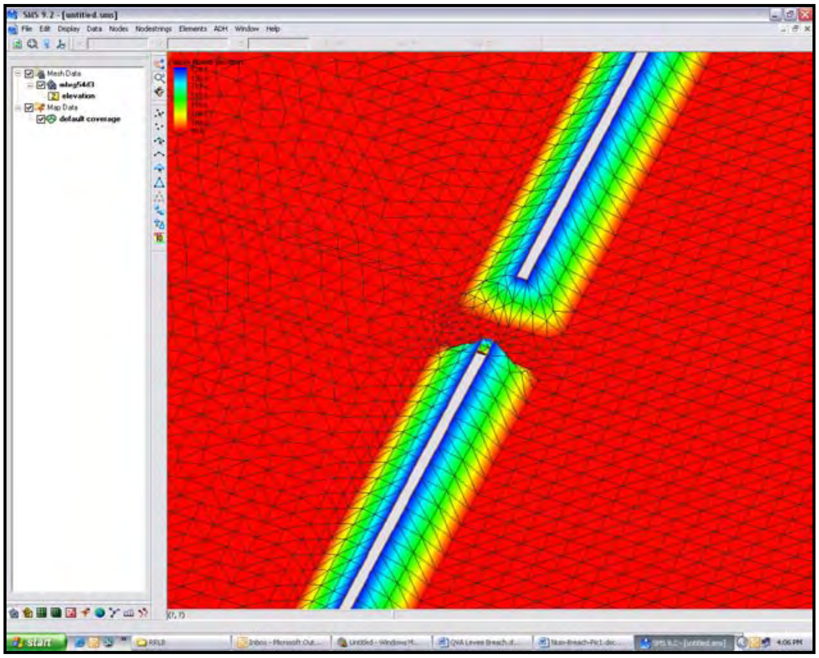


Figure 2.21. Close-up diagram of nodes in the vicinity of the idealized breach.

The simulation results show that a maximum velocity of about 22 ft/s occurred over a period of nearly 12 min into the simulation and was located about 50 to 250 ft downstream of the levee longitudinal centerline. Figure 2.22 shows a graphic of these results. Figure 2.23 shows a computation of the forces that would be exerted on a flat plate per foot of length for any similar object placed in the flow field at a given location.

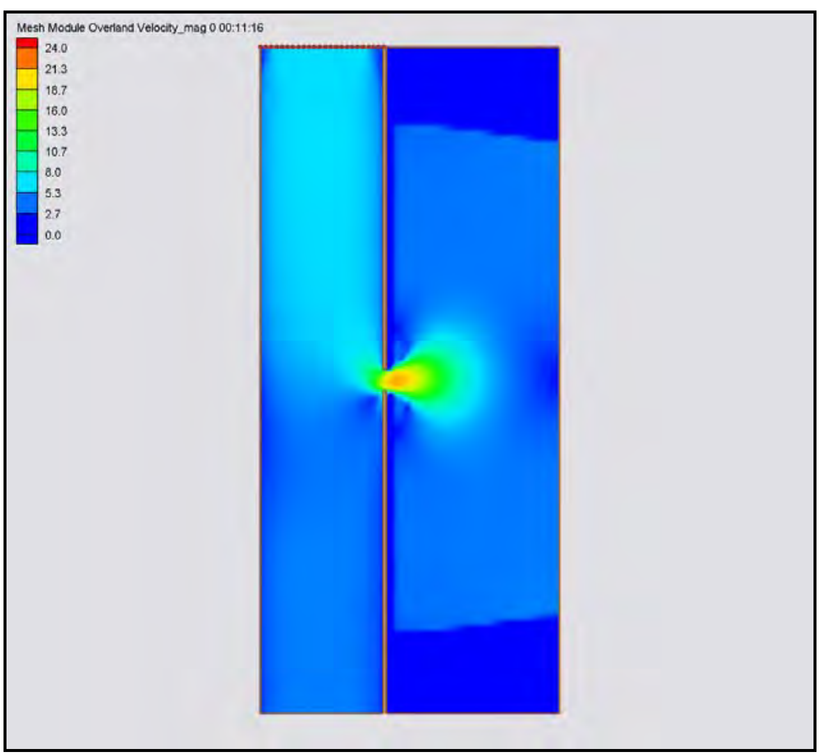


Figure 2.22. Velocity contours for flows modeled in idealized breach simulation.

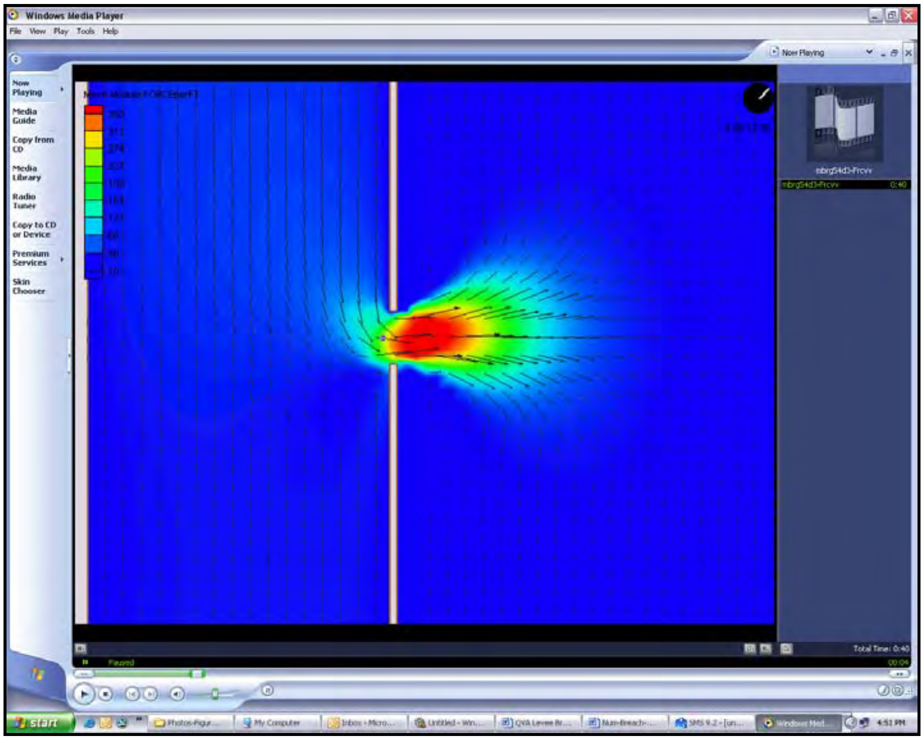


Figure 2.23. Modeled forces on a flat plate subjected to flows in vicinity of idealized breach.

These computations were made using Eq. (2.11):

$$F_d = \rho C_d A \frac{V^2}{2} \quad , \qquad (2.11)$$

where

F_d is the force acting on a hypothetical flat plate,
C_d is the coefficient of drag,
A is the area over which the force is acting.

The emphasis so far has been on the absolute highest velocities and forces occurring in the simulation. However, an object placed upstream of the breach would not necessarily encounter the highest velocities. It is also necessary to consider the direction of velocity in a lateral levee breach. As can be seen in Figure 2.24, the complete velocity vector can have a significant component in the streamwise direction that could seriously affect the effort to position the closure device. For example, at the centerline of the breach opening and about 150 ft from the levee crest on the river side of the breach, the component of velocity in the breach direction is 9.5 ft/s and the streamwise component is 3.3 ft/s. At the same distance from the levee crest but at the north abutment, the component through the breach is 2.6 ft/s and the streamwise component is 7.7 ft/s. So in moving a closure device a distance of only 100 ft in the streamwise direction and being 150 ft away from the levee crest, the velocity components change from 66% streamwise and 33 % breach direction to 66% breach direction and 33% streamwise. This shows clearly that the streamwise components of velocity cannot be ignored when considering the logistics of moving a closure device into place, which will become a very important topic when discussions for the concepts of operation for deployment are being developed.

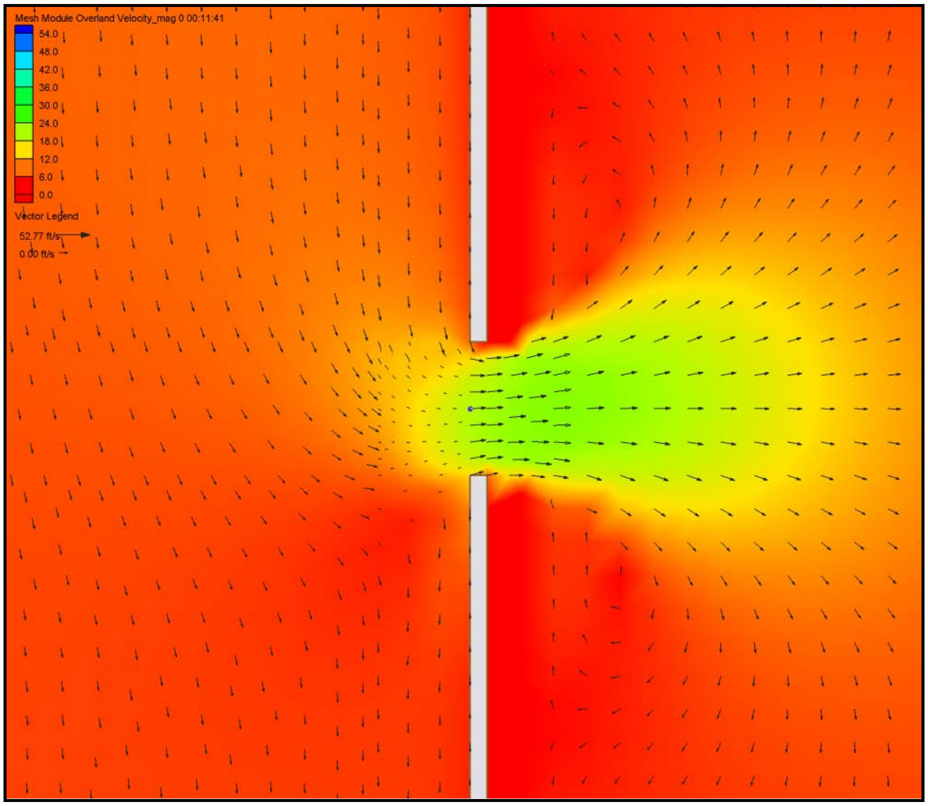


Figure 2.24. Force vectors (assuming a drag coefficient value of 2) for flow through an idealized breach.

2.7.2 Small-Scale (1 : 50) Model Concept Testing and Development

Although the project team relatively quickly developed some promising ideas about the potential for application of water-filled fabric components within rapid breach closure systems, we initially wanted to investigate a much wider range of alternatives before we focused only on this class of structure for RRLB solutions. For this reason, a small-scale modeling effort of the RRLB was conducted with the following considerations in mind:

1. generate new techniques for RRLB;
2. perform initial evaluation of proposed techniques for RRLB;
3. identify critical components of proposed techniques and possible deployment alternatives that might enhance the efficiency of the method and its success;
4. provide information on potential problems related to changes in the flow field, scour potential, or head differential; and
5. provide direct measurements of hydraulic parameters related to the forces.

The tests were also guided by information gleaned through a search of the current dam and levee breach literature and our compilation of important factors affecting breach formation, described in Chapters 3 and 4 of this report. With these considerations in mind the conceptual methodologies to test were initially listed according to the type of breach which might occur. At this scale, it is very easy to execute almost any option that can be imagined, so there was no attempt at this point to suppress concepts that might be impractical at larger scales. Instead, the small-scale model was used to provide a general learning process for the entire R&D team.

2.7.3 Small-Scale Modeling Flume Tests

An 80 ft long tilting flume was used to provide a simulated levee breach and flow for testing the various closure methods discussed above. The flume cross section is 3 ft wide and 1 ft deep. The facility has pumps that can supply up to 5.5 ft^3/s of flow capacity. To maintain a constant water level during the closure tests, a special side-weir-overflow device was constructed to fit within the flume. A simulated levee was constructed separately but made to fit exactly within the confines of this overflow device. The flume, overflow device, and simulated levee are all shown in Figure 2.25.

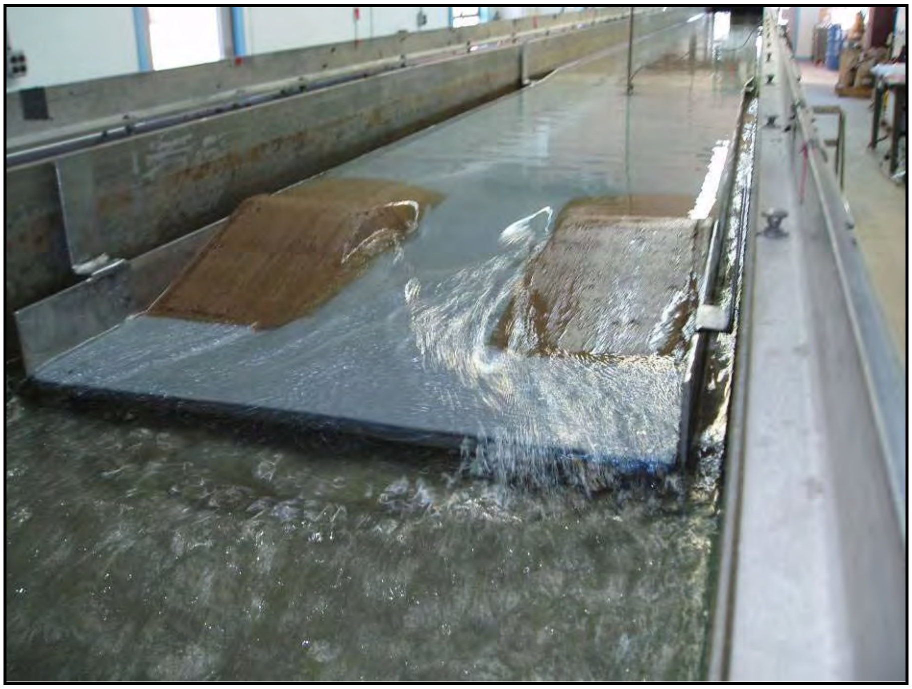


Figure 2.25. Photo of flow through small-scale (1 : 50) breach.

The model scale was 1 ft in the model to 50 ft in the prototype. The model levee was constructed to represent a Sacramento River Delta levee. Such a levee has a side-slope ratio of about 1 to 3 and is less than 20 ft in height. The modeled breach represented a 50 ft breach in the prototype. When flow calibration was complete, testing of closure methods could begin.

Four breach closure methods or combinations of methods were tested in the small-scale model during December 2007 and continuing into January 2008. They were placed into the simulated fully developed breach. The methods that were tested are described below.

1. Rectangular gated structure floated into place, with and without a tarp. Figure 2.26 shows the gated structure just after placement with gates still open. After gate closures, significant flow continued along the device perimeter. A tarp was used in combination with the gated structure to see how this would work in conjunction with it. The slow residual velocities allowed positioning of the tarp and actually helped move and seal the tarp against the structure. The result was an almost complete reduction of flow through the breach as shown in Figure 2.27.

Figure 2.26. Photo of gated structure test in 1 : 50 scale flume.

Figure 2.27. Test of a gated structure combined with a tarp showing almost no residual flow.

2. Barge floated into place with ballast on one side. A sealing fabric bag was added on the upstream side of the barge. Figure 2.28 shows the barge driven up the levee embankment with a simulated fabric bag placed against its upstream side. Significant flow reduction occurred.

Figure 2.28. Idealized barge floated into position on small-scale levee and ballasted with water.

3. Simulated cable net placed over the breach with tarp placed upstream of the breach. This is shown in Figure 2.29.

Figure 2.29. Photo of performance of a net anchored on either side of a breach combined with a tarp.

4. Geotextile fabric bags filled with water. For these tests, water-filled plastic bags were tested in anchored and unanchored configurations. Bags anchored on each end and filled in place are shown in Figure 2.30.

Figure 2.30. Photo of water-filled tube concept using two tubes anchored to either side of a breach in the 1 : 50 scale model basin.

Although the small-scale tests functioned as an extremely good learning tool, it soon became obvious that many of the concepts that were simple to deploy and functioned well at the 1 : 50 scale would likely not work well at larger scales. For this reason, we terminated testing in this basin and began preparation of a larger facility for subsequent tests.

2.7.4 Intermediate-Scale (1 : 16) Testing and Development

During the early phase of this project we investigated a wide range of ideas of potential value to rapid levee repair; but many of those ideas were soon recognized as either impractical or beyond the state of the science that exists today. We knew that we could not continue to devote extensive amounts of time to all of these different ideas and still meet our specified timelines for large-scale demonstrations. To assist us with our down-selection, we knew that it would be extremely useful to have a physical basin that was substantially larger than the 1 : 50 scale flume described previously in this report. An existing physical model facility within ERDC was identified as having the best potential for providing a suitable basin for such testing of RRLB concepts. This basin was modified from its initial purpose to provide a flow capacity consistent with a 1 : 16 scale along a levee section (Figure 2.31). The model levee was constructed at an undistorted scale and represents a levee with a crest elevation of 20 ft (note: all dimensions are reported in prototype scale unless otherwise stated), side slopes on both faces of 1 : 3 (vertical : horizontal), and a crest width of 16 ft. The breach was 80 ft wide with 2 : 1 side slopes the full depth of the levee. Both the model basin and the levee were constructed of a concrete cap poured over a sand fill with inset aluminum templates to ensure the accuracy of the bathymetry and topography.

Figure 2.31. Test basin for 1 : 16 scale physical model tests.

This model represents a fairly extreme test for breach closure because the breach is 80 ft wide and 20 ft deep, the still water level is 18.7 ft deep, and there is a significant riverine current flowing along, past, and through the breach (Figure 2.32).

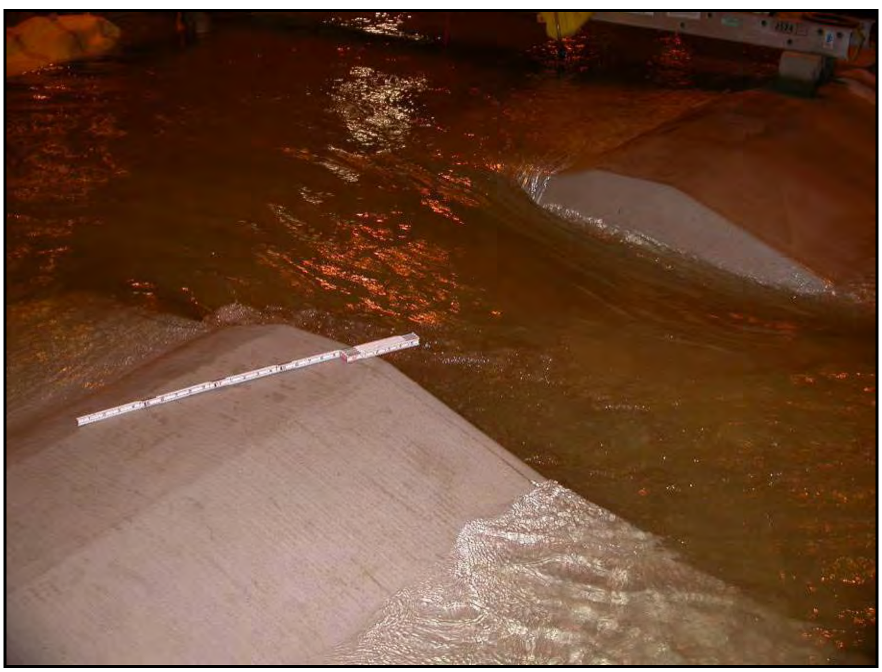


Figure 2.32. Flow through the breach with upstream depth at 18.7 ft.

Given our performance metrics for the RRLB system, analyses and tests indicated that the conventional methods of construction (using rigid members or fabric elements) did not appear to offer a good solution for stopping flows through a large breach. Unlike our tests in the 1 : 50 scale flume,

our tests in the 1 : 16 scale basin had to be much more focused. In this context, we decided to concentrate our effort on testing the new concept that was introduced at the end of Chapter 2. In this concept, the bending of the fabric tube (or more generally any fabric chamber) is not determined as much by the beam equation as by volumetric constraints within the tube.

To illustrate this concept, let us examine the deformation of a simple cylinder with flat, nondeformable rigid ends as shown in Figure 2.33. Hypothetically, if we fill this tube's interior volume with 100% water, the volume of the tube cannot be increased without a net increase in the surface area of the fabric (i.e., the fabric stretches). Conversely, if we attempt to bend this tube along the axis of the cylinder and the fibers along the outer curve are assumed to be incapable of stretching, such a deformation will require a buckling of the fabric on the interior curve, leading to a loss in volume. However, because water is essentially incompressible, the volume cannot decrease. Thus, the tube would resist deformation. Of course, actual fabrics do stretch and there will be some elongation (stretching) of the fibers along the outer curve. This elongation will be consistent with the total pressure force pushing outward on the surface of the fabric. This stretching of the fiber on the outside curve will allow a slight deformation in the overall shape of the cylinder, leading to bending. If the material can be stretched easily, an initially straight cylinder can be curved to any degree desired; however, if we use typical construction fabrics, the amount of stretching will be only about 5%–7% before the fabric fails (tears).

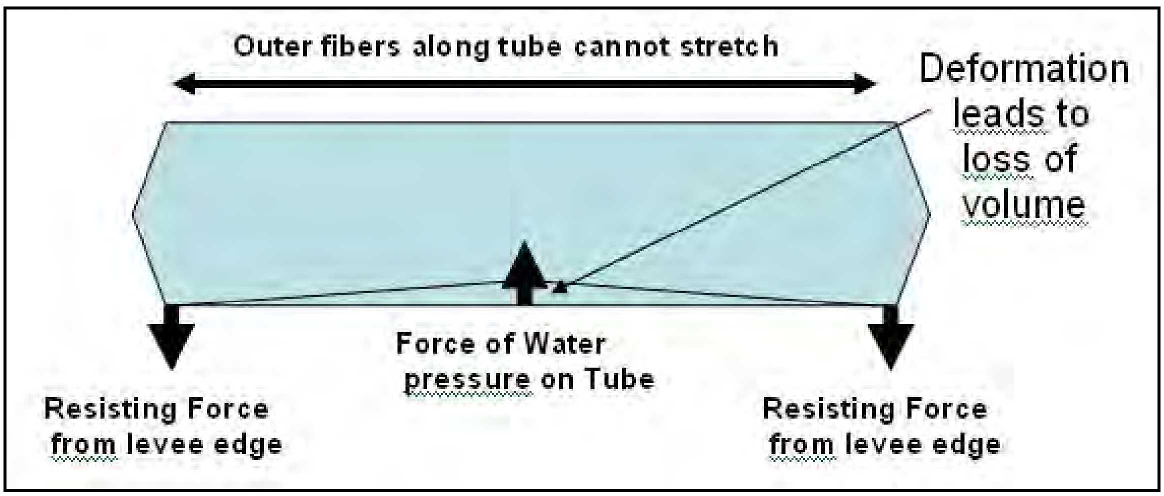


Figure 2.33. Idealized forces on a tube at a breach.

Because the results are not very dependent on the neglect of the 5%–7% stretch, we will neglect this and assume that the deformation can be represented in a simple sense by a situation in which we have only filled the tubes with water equal to 80% of the total of the tube. In this case, we can decrease the interior volume by 20% before any resistance to further deformation would occur. For the simplistic case where the tube shapes are maintained outside of the folded region, the bending would persist to the point where the volume within the shaded areas is equal to the 20% of the volume that was missing from the tube. Once this happens, the tube would again begin to resist further deformation.

In an actual fabric, the situation is a bit more complicated, but the basic concept remains the same—a water-filled fabric tube will begin to resist deformation once the interior volume is compressed to a 100% fill level. Figure 2.34 shows an idealized concept for a situation in which a tube has deformed to the point where the interior is 100% filled. Once the compression reaches this level, resistance to additional deformation will be proportional to the pressure distributed over the entire surface of the tube, which can be a very large force. Unlike the idealized case described here, the actual shape of the fabric will be a smooth curved surface which distributes the force and response

to the deformation over a wider area rather than the localized deformation shown in Figure 2.34, but the general principle will remain the same.

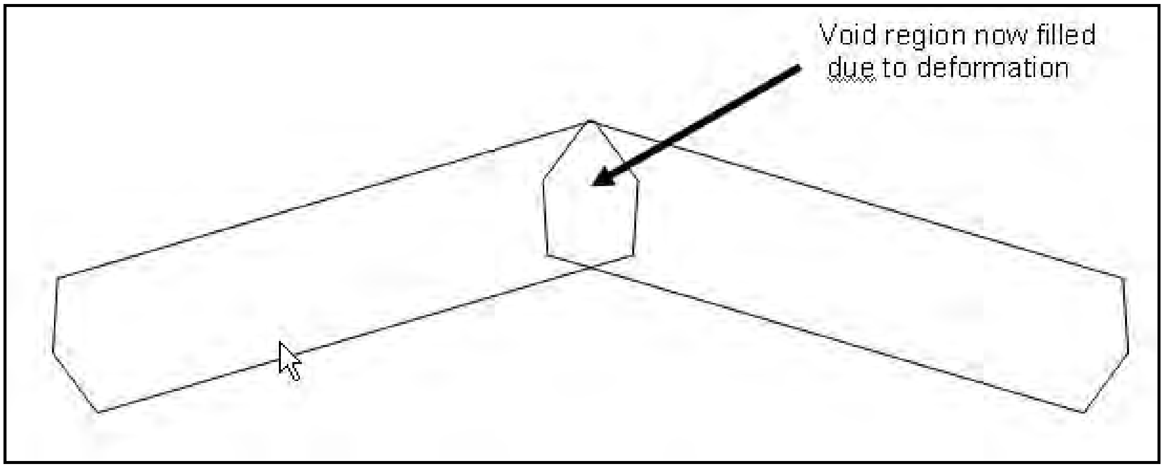


Figure 2.34. Deformation of a tube leading to loss of volume.

It is also possible to glean some general scaling principles from the simplified situation discussed here. One can see that the amount of deformation will depend on the ratio of the void volume to the diameter of the tube, giving us

$$\alpha = \tan^{-1}\left[\frac{\delta V_e}{\zeta \delta D_t^3}\right], \tag{2.12}$$

where

α is the angle subtended by the two cylinders in Figure 2.34,
δV_e is the volume of the tube that is empty,
ζ is a dimensionless coefficient of proportionality,
D_t is the diameter of the tube.

Although the exact point of failure will be determined by physical model tests, this equation should provide a good indication of the failure mode that occurs when a tube collapses onto itself to a point where it can pass through a breach opening. For breach widths less than twice the tube diameter, a conservative approximation for the failure point can be taken as 45° in estimates based on this equation. It is difficult to specify a functional relationship for this scaling behavior for wider breaches; so instead, we will simply rely on the physical model results.

It is likely that a numerical model can be developed to estimate the behavior of a tube undergoing deformations of the type being investigated here; however, this was well beyond the scope of the work conducted in the first year’s effort. A complicating factor is the relatively unknown coefficients of drag for the fabric sliding past the periphery of the breach. Analytical solutions for a situation such as this are also complicated by the fact that two factors must be considered in addition to the ability of the tube to avoid being swept through the opening: (1) the ability of the tube to conform to an irregularly shaped opening and to be able to seal the flow through it and (2) the ability of the tube to achieve and maintain sufficient freeboard to block the flow up to the water surface.

When a tube passes over a vertically raised perturbation (levee remnant) it can be seen that a resistance to passing over the remnant can be obtained by a combination of resistance to deformation and the weight of the water within the tube when it is lifted. Such a system is ideal for sealing wide, shallow breaches and might be very effective in helping prevent overtopping in areas such as Braithwaite, Louisiana, shown earlier in this report.

2.7.5 Intermediate-Scale Model Tests and Results

As noted earlier in this chapter, the breach tested in our model basin provided a substantial challenge because it represented an opening that was 18.5 ft deep and 80 ft wide. The primary successful tests of note for sealing this breach in the 1 : 16 basin involved variations with a single tube that would have a prototype length of 195 ft. Once we learned that the volume fill proportion for this tube could not be much less than 60%, essentially all tests managed to close all or close to all of the flow through the breach. Figure 2.35 shows a typical result obtained for a tube filled to a 60% capacity. As can be seen here, the performance is quite remarkable, with very little water passing through the breach after emplacement of the tube.

Figure 2.35. Deep-breach closure with large tube filled to 60% fill volume.

The deployment method in the tests in the 1 : 16 scale basin were all complicated (probably realistically) by the relatively fast currents passing by and parallel to the breach, similar to the case of a breach in a levee along a large river. It was noted that the tubes always had a very strong tendency to start rolling toward the breach once part of the tube came in contact with the bottom. As described in the next chapter, this tendency combined with the prolonged deformation of the tube plays a very positive role in reducing the dynamic forces on the remaining levee sections. However, a negative aspect of this rolling within the complicated flow field was that the tube sometimes began to twist differentially before it reached its final stopping position within the breach. This twisting appeared to provide additional avenues for water to pass through the breach after the tube was in place. Within the laboratory, it was relatively simple to execute techniques which could minimize this twisting, but the ability to accomplish this in similar situations at prototype scale will still have to be demonstrated.

Figure 2.36 shows the performance of the wide, shallow breach system in an application. This tube represents a tube that is 256 ft long and 8 ft in diameter at prototype scale. We originally performed a number of tests with a secondary (simulated air-filled) floatation tube attached to the 8 ft tube, but we found that removing the secondary tube improved our test results. Also during these tests, we realized that a major mistake had been made in the design of the facility which we did not have time to correct before the testing at Stillwater was to commence. The problem was that the wide, shallow breach vicinity was designed to also serve as a movable-bed test site. This colocation created continual problems with secondary flow through the submerged (remnant) levee throughout our tests.

Although it is very difficult to see in Figure 2.36 due to difficulties with the lighting and the secondary flow passing through the unconsolidated underlying materials, near-zero flow conditions were achieved in a significant number of tests. The problem with underflow was corrected in the

testing facility at Stillwater, and as will be seen, the results there confirmed our interpretation of the 1 : 16 scale results.

Figure 2.36. Performance of a very long water-filled fabric tube in a wide, shallow breach test in the 1 : 16 scale model.

3. RAPID REPAIR OF LEVEE BREACHES EFFORT: YEAR 2

3.1 Overview

The first year of the RRLB project produced a good indication of what could potentially be accomplished via some innovative concepts that were developed under this program. It also clearly focused attention on some elements of this work that needed further work before these new technologies could be accepted for real-world applications. The major issue that had to be addressed was proof of concept at full scale, because, although the RRLB tests were conducted in the largest facility in the United States for such testing, it was difficult to argue that tests with a breach width of 6–8 ft and a flow rate of 125 ft^3/s were truly representative of much wider breaches (60–80 ft wide) with flow rates in the thousands of cubic feet per second. In addition to the question of scale, another question that arose in the first year of testing pertained to how a temporary breach closure using a PLUG could be transitioned into a permanent repair. Thus, the second year of the RRLB effort focused on two primary issues: (1) development of a means to attain a proof of concept appropriate for full-scale breaches and (2) development of a means to transition from temporary breach closures to permanent repairs.

Work related to the proof of concept at full scale included four elements: (1) development of improved analytical tools for quantifying the forces on the PLUG and adjacent levees and for PLUG design, (2) measurement to verify analytical concepts, (3) transition from temporary to permanent repairs, and (4) selection of a location for and design of a physical facility for testing full-scale breaches.

3.2 New Analytics

In the first year of this effort, we concentrated on establishing a simple analytical framework for the ability of the PLUG to seal a breach and not pass through it. For the demonstration of this technology, the primary design issue was the hoop tension in the fabric. This tension is a function only of the water depth and tube diameter. For longer breaches a second factor, the total load across the breach, must be considered—in a fashion very similar to the load on a suspension bridge turned on its side. If we neglect friction along the bottom of the PLUG sealing a breach and recognize that the PLUG's own weight does not contribute significantly to the load on the PLUG, we can obtain a reasonable estimate for the load on a rectangular, cross-sectional area of a breach as

$$L_{tot} = \int_0^W \int_0^d p(x,z)dxdz \approx \int_0^W \int_0^d \rho gzdxdz = \frac{\rho g d^2 W}{2} \quad . \tag{3.1}$$

In the limit of a uniform, flexible fabric (i.e., one with no resistance to deformation perpendicular to the primary direction of the tension), we would obtain the classic solution for a catenary as the solution for the shape that the PLUG would take in response to a uniform load perpendicular to the breach. However, the PLUG's resistance to deformation is what makes it work, so instead, we shall treat the PLUG in terms of a simpler limit, that of a rigid solid across the breach connected by a very short fabric element to the sides of the breach.

In an average sense, this load is carried by1/2 of the circumference of the tube (the fibers on the outside of the curve), so a rough estimate of the strength of the fibers needed to carry 1/2 of the total load (assuming that both sides combine equally to carry the total load) would be

$$L_0 = \frac{L_{tot}}{\frac{\pi D}{2}} = \frac{\rho g d^2 W}{\pi D} \quad , \tag{3.2}$$

where

L_0 is the load that must be carried by a unit width of tube,
L_{tot} is the total load on the tube,
D is the tube diameter.

And if we assume that the diameter of the tube is scaled to be some fraction larger than the water depth (as required to block the flow), this becomes

$$L_0 = \frac{\rho g d W}{\gamma \pi} \quad , \tag{3.3}$$

where

L_0 is the load that must be carried by a unit width of tube,
γ is the ratio of the tube diameter to the depth.

The tension required to hold this load will depend on the orientation of the fibers at the intersection with the side of the breach relative to the direction of the load:

$$\tau_0 = \frac{L_0}{\cos(\theta)} \quad , \tag{3.4}$$

where

τ_0 is the tension per unit width in the fibers that must be carried,
L_0 is the load that must be carried by a unit width of tube,
θ is the angle between the load direction and the fiber direction at the breach edge.

Experiments suggest that the PLUG forms an angle at the point of intersection with the breach that is about 40°–60° relative to the levee. If we take the lower limit of this deformation angle to be about 40°, we have a simple estimate for the required strength of material used in the PLUG for a given size application:

$$L_0 = \frac{\rho g d W}{\gamma \pi \cos(40^o)} \quad , \tag{3.5}$$

where

L_0 is the load that must be carried by a unit width of tube,
γ is the ratio of the tube diameter to the depth.

Because Eq. (3.5) includes a dependence on the width of the breach, it will be larger than the hoop tension for most breach geometries expected in nature.

Another failure mode of the PLUG can occur when the PLUG rolls over the top of the sides of the breach. This failure mode is described in some of the tests contained in Ward et al. (2011) in situations where the water level approached the top of the levee. In this situation, the resistance of the PLUG to this failure mode relates to the weight of water that must be lifted above the local still water

level for the PLUG to pass over the top of the levee. Increased water fill within the PLUG decreases its ability to deform and forces it to act more like the long-breach system described in Chapter 2 of this report. Additional work is needed to improve our understanding of the PLUG deformation before we can obtain good estimates of this behavior; however, such work is beyond the scope of the present effort.

3.3 Measurements to Verify Analytical Concepts

A companion report to this one, *Laboratory and Field Tests in Support of Rapid Repair of Levee Breach Study* by Ward et al. (2011), contains a description of all work pertinent to this topic conducted as part of the overall RRLB effort and will not be repeated here.

3.4 Transition from Temporary to Permanent Repairs

A critical question that was voiced at the end of the year 1 demonstration was "how can the PLUG be replaced during the transition to permanent repairs?" Because the PLUG might be situated in areas where its removal would lead to widespread flooding even if it were removed after the flood crest had subsided, it is obvious that its removal cannot be done before some additional flood-prevention structure has been emplaced. Typically, a cofferdam is constructed in such situations before the temporary repairs are removed; however, because cofferdams often can take up to several months to complete, this would mean that the PLUG would have to function, probably over a relatively wide range of water depth and exposure to debris impacts, for an extended period of time.

After discussions and in conjunction with ongoing work for the rapid repair of damaged/ malfunctioning navigation structures, we developed a concept for a new class of expedient fabric structure to address this concern. Similar to the situation in construction with solid elements, the concept was to use the shape of the structure to help carry critical loads and to provide space in front of the breach for PLUG removal and completion of permanent repairs. The obvious solution to both of these needs was seen as an arch-shaped structure, which soon became designated as the Arch-Shaped Reusable Cofferdam & Hydrodam (ARCH). As the ARCH is somewhat peripheral to the main thrust of the DHS-SERRI funding in this project, only a cursory description of the technology and testing results will be given here.

Tests at small scales (about 1 : 16) suggested that the ARCH was capable of stopping flows in fast flowing situations typical of navigation structure repairs. Figure 3.1 shows a test in which a two-tube (one stacked on top of the other) system was used to block the flow. Essentially all of the flow through the gate was blocked and the only water on the downstream side of the ARCH was due to water flowing past the sides of the ARCH.

Figure 3.1. Successful laboratory tests of a two-arch system for blocking flow through a simulated "gate" opening. Actual distance spanned by the ARCH in these tests is about 1.5 ft.

This concept also proved capable of providing an expedient dam across relatively large expanses of water (Figure 3.2). Given the success at this scale, a decision was made to perform tests in the fall of 2009 at Stillwater at a much larger scale.

Figure 3.2. Successful small-scale test of a single-arch system for blocking flow across a large open area. The actual distance spanned by the ARCH in this test is about 25 ft.

Testing of the ARCH in Stillwater consisted of two parts: (1) a test of its capacity to serve as an enabler for rapid transition from the PLUG to permanent repairs at a site and (2) a test of its ability to serve as an expedient dam to block water across a relatively large expanse of water. Figure 3.3 shows some early results for the first of these tests. In this test, the “web” across the interior of the ARCH, seen in the right-hand panel, is to prevent the deformation of the ARCH. During subsequent tests, it was found that the web alone was not sufficient to prevent all unwanted deformation; furthermore, such deformation, if uncontrolled, could lead to structural failure. For this reason, the ARCH was modified to have an interior “ballast tank” at the apex of the ARCH. Figure 3.4 shows the result of this modified ARCH structure during a test in Stillwater in November 2009. Figure 3.5 shows the same modified ARCH acting to stop water from flowing past it into a 40 ft wide channel. Water depths at the apex of the ARCH tests shown in Figures 3.4 and 3.5 were about 3.5–4.0 ft.

Figure 3.3. Early tests of the ARCH showing its ability to seal the area around a breach to allow the PLUG to be emptied and removed before permanent repairs commence.

Figure 3.4. Test of the modified ARCH system at Stillwater, Oklahoma, in November 2009. The ballast tank seen behind the apex of the ARCH improved its ability to resist deformation under very high water levels.

Figure 3.5. Test of the modified ARCH at Stillwater, Oklahoma, in November 2009 showing its ability to block water from flowing into a channel about 40 ft wide with a water depth of about 4 ft.

Deployment of the ARCH was typically accomplished by first filling the ARCH with air. This made the ARCH very easy to handle manually in the water and enabled it to be easily maneuvered

into position. In step two of the deployment, the air in the water was allowed to flow out of valves at the top of the ARCH while water was being added to it. The system used in the Stillwater tests was relatively crude and manual, but the ARCH at this scale could still be emplaced and functional within 1 h. It is envisioned that an automated system for removing the air and replacing it with water could allow a much larger system to be emplace in 1–3 h.

3.5 Development of Full-Scale Test Facility for Full-Scale Proof-of-Concept Testing for PLUG

Considerable effort was spent in the summer and autumn of 2009 developing low-cost designs for a full-scale test facility and searching for a suitable location for it. A very strong case was made for locating the facility in the Sacramento area, but we failed to find a suitable site in this area for which all the environmental permits for construction could be obtained in a short period of time, as required to complete the DHS proof-of-concept testing. Consequently, in mid December, following attempts to find potential sites in the vicinity of Stillwater, Oklahoma, and Oxford, Mississippi, we decided to proceed with construction in Vicksburg, Mississippi, on the ERDC grounds. The primary motivations for this decision were (1) environmental permitting could be obtained relatively quickly, (2) property was available with the proper slope and of sufficient size to build the facility, and (3) time constraints for the final tests eliminated the opportunity of additional searching before commencing construction.

During the same interval that the search for a suitable full-scale test facility was occurring, the design of the facility evolved into a three-basin concept as shown in Figure 3.6.

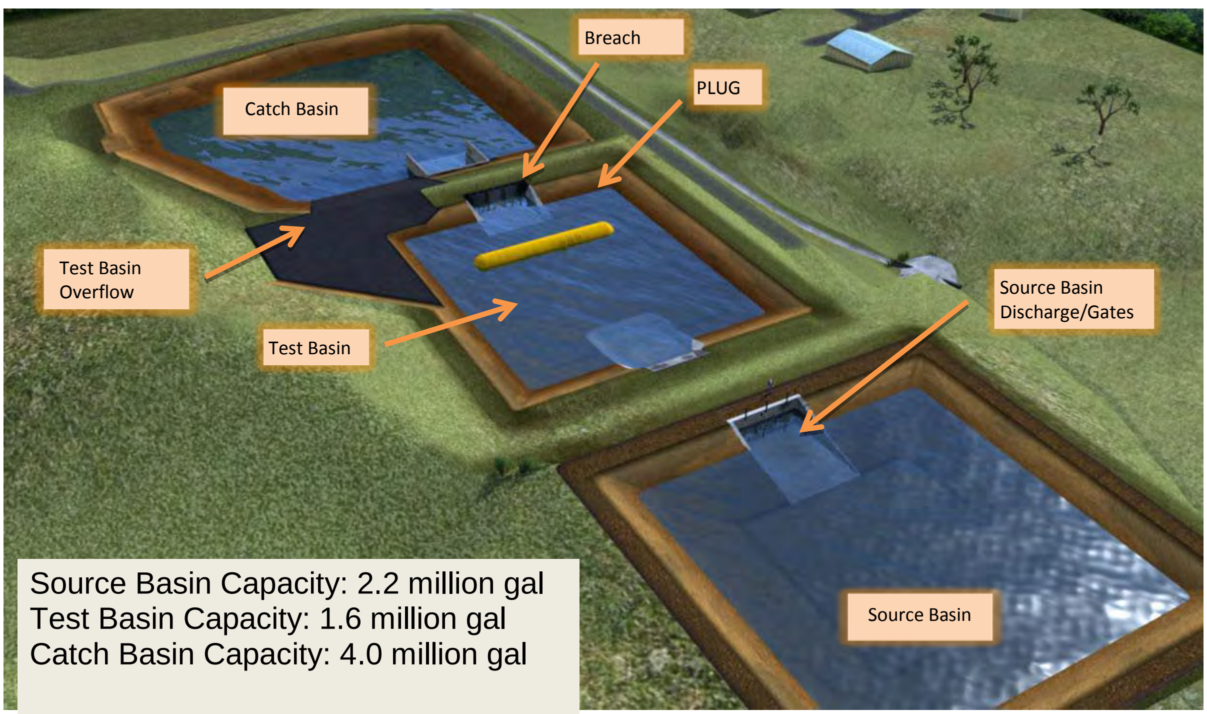


Figure 3.6. Artist's rendition of the three-basin full-scale test facility.

The design goal of this facility was to obtain a flow rate of at least 2,000 ft^3/s through a 40 ft breach, representing an increase of a factor of 16 in the flow capacity over the maximum flow rates we could achieve in Stillwater. Although this flow rate can only be maintained for about 3 min, the time for the PLUG to reach the breach and seal it is much less than this, so the lack of a continuous

flow capability was not considered to be a major problem in this design. Details of this design and full-scale proof-of-concept testing are presented in the next chapter.

4. RAPID REPAIR OF LEVEE BREACHES EFFORT: YEAR 3

4.1 Overview

The final year's effort on the RRLB project was divided into two major tasks: (1) design and construction of the RRLB full-scale facility and (2) proof-of-concept testing of a full-scale PLUG. Time and budget constraints were difficult to work with, but in general, the results were extremely positive. A companion report by Oceaneering International, Incorporated, provides additional technical details on the design and construction of the full-scale test basin and testing of the full-scale PLUG in this basin (Oceaneering International, Inc., 2011).

4.2 Full-Scale Facility Design

The test facility was designed to accommodate the testing of full-scale RRLB technologies. We decided that the PLUG would be the only test article tested at full scale. The performance requirements of the test model to support PLUG testing were as follows.

- 40 ft wide breach measured from mid height of the breach
- 8 ft water depth at breach
- Source basin to accommodate a flow rate of 2,000 ft^3/s for one test
- Average flow rate of 1,000 ft^3/s for 5 min
- Support of one test per day; able to replenish water volume to repeat test in 20 h
- Flow control gates open at 1 in./s to support desired flow rate

The design of the test model consisted of three earthen basins using gravity to convey water through each basin. A pump was used to replenish the water in the source basin after each test. Slide gates were located at the source and test basin inlet structures to control the water flowing between basins. The water was conveyed through the embankments with 54 in. diameter steel pipe. Areas of high water flow around the structures were reinforced with low strength concrete to mitigate erosion. A 2 ft thick layer of clay on the bottom and slopes of each basin was used to prevent seepage. The levee crown around each basin was a minimum of 10 ft for easy access. Road surfacing was used only in areas where it was anticipated that vehicles might be needed. Design details of each basin are discussed in the following sections.

As noted above, a requirement of the test model was for the breach to be 40 ft wide and to pass a flow rate of 2,000 ft^3/s of water through it during the PLUG test. The breach area was designed as a concrete structure that could hold up to repeated testing and water flow. The 40 ft width was measured from the mid height of the breach and the inside slopes were 1 : 2, as designated by ERDC. Another feature of the breach was a 10 in. pipe that originated in the test basin and terminated in the concrete breach wall that allowed for the breach area to be flooded with water when the PLUG was in place in the breach. By filling this area with water after it had been vacated following a successful test, the pressure on the PLUG was equalized by having the same water level on either side. This equalization allowed the PLUG to be maneuvered away from the breach easily, reducing reset time for the next test. A gate valve on top of the south levee slope was used to control the flow of water through the equalization pipe.

As the breach was designed to pass 2,000 ft^3/s, the structure behind the breach that conveyed the water into the catch basin had to also. Three flow control gates were used to impound the water in the test basin before a test and then raised to allow water to flow through the breach into the catch basin. When the gates were raised, the water flowed into six 54 in. diameter pipes that carried it into the catch basin. A 10% slope of the pipe was used to achieve the vertical drop that provided adequate flow. The flow control gates were designed to open at a rate of 1 in./s. This allowed the gates to open

fast enough that a minimal amount of water would be lost into the catch basin before the flow rate reached 2,000 ft^3/s.

The overall dimensions of the test basin were designed to be able to accommodate a PLUG that could seal a 40 ft breach. With the PLUG being 100 ft long, the test basin was designed to be 120 ft long at the base. This gives ample room for the PLUG but is narrow enough that the PLUG cannot drift too far off center during deployment. The length of the test basin was designed such that the turbulence from the influx of water from the source basin to maintain the water level in the test basin has minimal affect on the PLUG. Consideration was also given to the fact that a variety of deployment scenarios would be tested in relation to how far the PLUG was from the breach. The final length of the base of the test basin was 150 ft. The slopes on all four sides were 1 : 3 with the height of the levee walls being 12 ft. The 12 ft high slopes ensured that in any condition, the water coming into the test basin could not overtop the levee. Other features of the breach include a safety handrail on top of the levee directly behind the breach and a wide access area behind the breach that has road surfacing material on which a crane could be placed if needed during testing.

On the southeast corner of the test basin, an overflow spillway was incorporated to allow water a passage into the catch basin while the PLUG was emplaced in the breach. This is designed to be able to pass at least 2,100 ft^3/s of water without overtopping. The entire spillway area was covered with flowable fill material to prevent erosion. The invert of the spillway was designed to be 5.5 ft below the top of the levee slope. That only left the test basin capable of holding 6.5 ft of water before it would flow over the spillway. This was done as a result of the discovery in recent 1/8-scale model testing that the PLUG could fail if the water level in the test basin approached 10 ft. Lowering the spillway would allow a larger volume of water to pass through the spillway if the water level in the test basin rose too quickly. To accommodate the lower spillway and still be able to have at least 8 ft of water in the test basin, an inflatable weir could be installed in the spillway. The weir would effectively increase the depth of water that the test basin could hold but could be rapidly deflated so that a larger volume of water could quickly exit the test basin via the spillway if needed.

4.2.1 Source Basin

The purpose of the source basin is to supply water to the test basin during a PLUG test so that the volume of water coming into the test basin matches the volume leaving the test basin through the breach. By maintaining the water level in the test basin as a test is in progress, the desired flow rate is sustained. The source basin was sized so that 1,000 ft^3/s of water could be discharged for 5 min and a flow rate of 2,000 ft^3/s could be attained for the duration of a PLUG test.

The above requirements led to three critical design points for the catch basin: total capacity, water depth, and flow control gate speed. We determined that for a flow rate of 1,000 ft^3/s for 5 min, the source basin would need to have a capacity of 2.2 million gal of water. This is achieved with a basin that is 94 ft by 94 ft at the base with 1 : 3 sloped embankments and a water depth of 15 ft. To maintain a 2 ft freeboard in the basin at all times, the basin was designed to be 17 ft deep. We determined that to achieve a flow rate of 2,000 ft^3/s into the test basin a combination of a total of 20 ft of hydraulic head and flow control gates that would open at 1 in./s would suffice.

4.2.2 Catch Basin

The catch basin was designed to hold the combined total of water in the source basin and test basin, about 4 million gal. The depth of the catch basin in relation to the earthen levee slopes was 12 ft. The catch basin contained a stilling basin to dissipate the energy of the water coming from the test basin. Flowable fill material was used around the stilling basin and at the base of the test basin spillway to prevent damage from erosion. Other features included a safety handrail around the top of the stilling basin and a concrete pad on which the submersible pump would be placed.

Two features were used to prevent the catch basin from overflowing. A 24 in. riser pipe was placed at 9 ft above the basin floor. If the water level in the catch basin would reach this height, it would drain down the riser pipe into existing drainage on the property. An emergency spillway was excavated into the embankment a foot above the top of the riser pipe to further ensure that if flooding conditions existed, water would be contained to natural drainage areas. A gate valve that could be opened to drain water from the basin directly into the natural stream was also included.

The water return system replenishes the source basin with water that has been collected in the catch basin. We determined that to be able to perform several tests per day the return system should be able to completely refill the source basin in 20 h. The source basin has a volume of 2.2 million gal of water, meaning the pump used for the water return must have a capacity of at least 1,833 gal/min. The pump selected has a discharge size of 12 in. Piping was used to carry the water from the pump back to the source basin. The pipe used was 14 in. diameter, leaving a margin in case a larger pump was desired for future tests.

4.2.3 Piping Between Basins

The pipe that conveys water from the source basin into the test basin and from the test basin into the catch basin is 54 in. diameter Max Flow Spiral Rib Pipe from Southeast Culvert, Inc., in Auburn, Georgia. The pipe is aluminized steel that has a life of 75 years minimum when installed in the recommended environment. At each of the two locations, there are six runs of pipe that extend from the inlet structure to the outlet structure. In all, 1,017 ft of pipe was used between the two locations.

4.2.4 Control Gates

The flow control gates for the source basin and test basin structures were designed and fabricated by Golden Harvest, Inc., of Burlington, Washington. The main components of the gates include a guide rail, gate head, stem, stem guide, stem coupler, wall bracket, and various seals. The guide rails contain ultrahigh molecular weight polyethylene seating faces, and the gates contain neoprene face seals for low leakage. The gate actuators are hydraulic cylinders. The cylinders have a 5 in. bore and a 2 in. diameter rod with a 60.25 in. stroke. Hydraulic pressure to the cylinders was provided via a hydraulic power unit (HPU) that was fabricated by Hydro/Power of Jackson, Mississippi. Two HPU’s were used, one to power the source basin gate actuators and one to power the test basin gate actuators. The HPU’s contain an electric-powered 2.75 in.^{3} displacement pump which can deliver a flow of 19.1 gal/min at 1,750 rpm, assuming 92% efficiency.

4.2.5 Power

Power was installed at the test model by ERDCs’ Department of Public Works (DPW). The HPU’s for the gate actuators demand three-phase 480 V power to operate. DPW installed two utility poles to bring the power overhead from an existing pole to the site. One pole was placed on top of the levee embankment at the southwest corner of the source basin. The other pole was placed to the west of this new pole leading to the existing pole. From the pole at the source basin, the power lines were run underground in conduit to the HPUs at the source basin and test basin. DPW terminated the power lines to the HPUs. A disconnect switch for the power service is located at the pole on top of the source basin levee.

4.3 Construction

Construction work began at the test model on June 14, 2010 (Figure 4.1). Construction of the test model comprised three major activities, earthwork, structures, and subsystem installation.

Construction was substantially complete on October 25, 2010, following two successful acceptance tests. Figure 4.2 shows an aerial photograph of the full-scale test facility taken during the PLUG demonstration on December 15, 2010.

Figure 4.1. Early stage of construction July 2010.

Figure 4.2. Aerial photograph of full-scale test basin, December 15, 2010.

4.4 Full-Scale Plug Testing

The first test of the PLUG in the newly constructed test basin was conducted on October 22, 2010. Because the source basin was still undergoing some remedial work at this time, the test was conducted with only the water in the test basin available. The water level was set at 8 ft and the PLUG was presumed to be filled to a 65% fill level. The initial position of the PLUG was 20 ft in from of the breach when the breach gates were opened. As the PLUG approached the breach, it

became obvious that there was insufficient water volume in it to prevent it from passing through the breach; consequently, the PLUG did not stop at the breach, but with the ends folded completely back, it passed through the breach and struck the breach gate protection structure and the breach gates. Although the breach gates were not damaged, considerable damage to the breach protective structure occurred, and the PLUG was torn in three different locations. Figure 4.3 shows the aftermath of this test.

Figure 4.3. First PLUG test culminated in a failure, with substantial damage to the gate protection system and tears to the PLUG.

Because the failure in the PLUG, as documented in videos of the event and forensic analysis of the locations of damage to the PLUG, was not related to a deficiency in the fabric, the two obvious reasons for insufficient water volume are (1) the material stretched to a point where the volume fell under the critical amount needed to prevent complete folding or (2) the instrument used to measure the pumping rate into the PLUG was inaccurate. As the fabric in question could stretch up to 7% before breaking, a conservative estimate of the effect of stretching would be to reduce the effective fill from 65% to about 61%. Although this is somewhat on the low side of the optimal fill percentages used in the Stillwater, Oklahoma, tests, it did not seem to be sufficient to have caused the observed failure.

After repairing the PLUG locally and completing some remaining construction-related tasks for the full-scale basin, testing resumed on October 29. Because we were not sure that our fill measurements were accurate, we approached the issue of percentage fill with a good deal of caution, starting from a fill rate of 74% and slowly reducing it to achieve optimal results. Figures 4.4 and 4.5 show the results from a test with 70% fill. The closure was essentially 100% with an estimated 1 ft^3/s or less passing the PLUG and through the breach. This test was initiated with the PLUG located 40 ft in front of the breach, a water depth of 8 ft, and water flowing from the source basin into the test basin. Additional details of testing can be found in the Oceaneering International, Inc., report.

Figure 4.4. Successful test of the PLUG which achieved almost 100% stoppage of flow. The view in this figure is from the downstream side of the breach, with the water that is being held back from the breach shown in the upper right.

Figure 4.5. Photograph of the same test result as shown in Figure 4.4, except that it is taken from the opposite side of the breach. The curvature of the tube can be clearly seen along with rotational distortion in the longitudinal straps.

4.5 Concepts of Operation for PLUG

No funding for testing of full-scale deployment was provided during this project. Although tests of deployment were conducted in Stillwater, Oklahoma, these should be considered extremely provisional. Some options for deployment are shown and discussed on pages 79–83 of the Oceaneering International, Inc., report; however, these are still only notional at this time.

It is envisioned that there are two primary situations in which the PLUG or variations on the PLUG might be used: (1) at a primary breach site on a levee and (2) at a secondary location within an area undergoing flooding. The first situation is typical of those tested during this 3-year SERRI project and is what most people commonly think of when they picture the deployment of the PLUG. However, the second situation might be very critical in many areas around the United States where large basins are being flooded. A classic example of this situation would be flooding in the vicinity of Sacramento, where a basin can take several days to completely fill. In this case, if openings along

natural and manmade barriers (e.g., along railroads and major highways) are closed, substantial damages can be avoided.

An interested variation on the deployment of the PLUG would be to use it to seal the sides of a breach to inhibit further breach widening. Theoretically, it should be very possible to do this by allowing a sufficient overlap of the nonbreached levee before the PLUG wraps around the end of the breach. This has been demonstrated at laboratory scales but has not been demonstrated at larger scales. This capability could be critical in situations where the breach is very rapidly growing through erodible material, such as would be expected in many parts of Lake Okeechobee and the Sacramento area. Such a capability could save many millions of dollars per breach over the current alternatives, which allow breaches to grow until the water levels essentially equilibrate on both sides or the flooding subsides. This alternative also suggests that, for major breaching, it might be best to seal the breach edges before proceeding to attempt to seal the breach itself.

5. CONCLUSIONS

5.1 Accomplishments

This 3 year SERRI-HSARPA effort has clearly demonstrated that innovative structures for RRLB are possible. Furthermore, this effort has shown that lightweight fabric structures might play an important role in revolutionizing the way in which threats to life and property from levee breaches could be handled. The first year of this effort was quite broad and showed that fabric structures could provide a wide range of benefits in situations with potential flooding hazards. Following the success of this first year, the focus shifted from a broad research perspective on potential tools for mitigating flooding hazards to research focused on establishing a proof of concept for the ability of a PLUG to seal a full-scale breach.

The following work elements have been successfully completed under this research effort:

1. a conceptual investigation of various alternatives for breach mitigation,
2. a rough theoretical framework for design of fabric structures for breach mitigation,
3. a demonstration of a successful system for protecting exposed levees during overtopping,
4. a demonstration of a very long breach closure system,
5. a demonstration of the PLUG’s capability to seal a 7 ft wide breach,
6. a conceptual framework for PLUG deployment,
7. a method of transitioning from a temporary PLUG to permanent repairs,
8. a design of a full-scale facility for testing the PLUG, and
9. a demonstration of the PLUG’s capability to seal a 40 ft wide breach.

5.2 Recommendations for Future Work

The PLUG has now undergone extensive proof-of-concept testing at a scale typical of real-world problems. However, no testing of the full-scale deployability has been funded; consequently, it is difficult to argue that this system has been shown to work in real-world situations. Some of the reluctance in moving forward with the PLUG technology is undoubtedly due to the fact that levees are owned and operated by many different organizations within the United States; therefore any unified approach to dealing with levee breaching problems is almost impossible to attain. However, some of the reluctance may also be due to the “wishful thinking” that there are existing methods that can work in the same situations in which the PLUG is intended to function. In fact, in several meetings over the last 3 years it has been argued that at least two solutions exist which can work as well as (or perhaps even better than) the PLUG in such situations. These two solutions are (1) filling the breach with large rocks and (2) sinking a barge to seal a breach. However, our team is not aware of any successful applications of either of these methods to seal a breach within a time frame commensurate with that of the PLUG technology.

Even as recently as December of 2010, at the Canal Del Dique breach in Colombia, a large number of vessels combined with substantial supporting ground assets attempted to seal a breach that was about 150 ft wide when workers began their attempts to seal it. Instead of being sealed, the breach continued to grow, essentially uncontrolled, for many days. Similarly, the sealing of the Jones Tract Breach in California was accomplished only after breach growth had abated after many days. In a similar vein, we could find no documented cases in which a barge had been successfully deployed to seal a breach. Besides the obvious problems with getting heavy equipment, barges, and heavy fill materials to a breach site, the technical merit of these approaches is essentially unverified. We would welcome attempts to prove/demonstrate the effectiveness of these older technologies, or other more innovative technologies, in the new full-scale test basin at ERDC in Vicksburg.

6. REFERENCES

Bromwell, L.G.; Dean, R.G.; and Vick, S.G. (2006). *Report of Expert Review Panel Technical Evaluation of Herbert Hoover Dike Lake Okeechobee, Florida.* Prepared for South Florida Water Management District. BCI Engineers and Scientists, Lakeland, Florida.

Bulson, P., "Design Principles of Pneumatic Structures," *The Structural Engineer*, 51(6) (June 1973).

Cavallaro, P.V.; Sadegh, A.M.; and Quigley, C.J., "Contributions of Strain Energy and PV-work on the Bending Behavior of Uncoated Plain-woven Fabric Air Beams," *Journal of Engineered Fibers and Fabrics*, 2, pp. 16–30 (2007).

Hanson, G.J.; Cook, K.R.; and Hunt, S., "Physical Modeling of Overtopping Erosion and Breach Formation of Cohesive Embankment," *Transactions of the ASABE*, 48(5), pp. 1783–1794 (2005).

Interagency Performance Evaluation Task Force (IPET, 2006). *Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System*, draft final report of the Interagency Performance Evaluation Task Force, U.S. Army Corps of Engineers report, 259 pp.

Main, J.A.; Peterson, S.W.; and Strauss, A.M., "Load-Deflection Behavior of Space-Based Inflatable Fabric Beams," *J. of Aerospace Engineering*, 7(2), pp. 225–238 (1994).

Mount, Jeffrey, and Twiss, Robert (2004). *Subsidence, Sea Level Rise, Seismicity in the Sacramento-San Joaquin Delta: Report to the Levee Integrity Subcommittee of the California Bay-Delta Authority Independent Science Board*.

Oceaneering International, Incorporated (2011). *Rapid Repair of Levee Breaches—Full Scale Test Model and Testing Summary*, TM305316, 145 pp.

Scientific Assessment and Strategy Team (SAST, 2007). *Science for Floodplain Management into the 21st Century, A Blueprint for Change*, Part V, Interagency Floodplain Management Review Committee.

URS Corporation/Jack R. Benjamin and Associates, Inc. (2007). *Delta Risk Management Strategy Phase 1; Topical Area: Levee Vulnerability*, Draft 2, Technical Memorandum, prepared for California Department of Water Resources.

Von Thun, J.L., and Gillette, D. R. (1990). *Guidance on Breach Parameters.* Internal Memorandum, U.S. Dept. of the Interior, Bureau of Reclamation, Denver, 17 pp.

Wahl, T.L. (1998). *Prediction of Embankment Dam Breach Parameters—A Literature Review and Needs Assessment*, Dam Safety Rep. DSO-98-004, U.S. Dept. of the Interior, Bureau of Reclamation, Denver.

Ward, D.L.; Resio, D.T.; Kleinman, A.G.; Boc S.J.; and Fowler, J.E. (2011). *Laboratory and Field Tests in Support of Rapid Repair of Levee Breach Study*, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, 60 pp.

Wielgosz, C.; Thomas, J.C.; and Le Van, A., "Mechanics of Inflatable Fabric Beams," *ICCES*, 5(2), pp. 93–98 (2008).

Zech, Y., and Soares-Frazao, S., "Dam-Break Flow Experiments and Real-Case Data; A Database from the European IMPACT Research," *Journal of Hydraulic Research*, Vol. 45 Extra Issue, International Association of Hydraulic Engineering and Research (2007).

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